SEISMIC RISK ASSESSMENT OF RC FRAMED VERTICALLY IRREGULAR BUILDINGS

MONALISA PRIYADARSHINI

DEPARTMENT OF CIVIL ENGINEERING
NATIONAL INSTITUTE OF TECHNOLOGY
ROURKELA 769008
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MONALISA PRIYADARSHINI

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Prof. ROBIN DAVIS P

DEPARTMENT OF CIVIL ENGINEERING
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CERTIFICATE

This is to certify that the thesis entitled "**SEISMIC RISK ASSESSMENT OF RC FRAMED VERTICALLY IRREGULAR BUILDINGS"** submitted by **Monalisa Priyadarshini** in partial fulfilment of the requirement for the award of **Master of Technology** degree in **Civil Engineering** with specialization in **Structural Engineering** to the National Institute of Technology, Rourkela is an authentic record of research work carried out by her under my supervision. The contents of this thesis, in full or in parts, have not been submitted to any other Institute or University for the award of any degree or diploma.

Project Guide

Prof. ROBIN DAVIS P

Assistant Professor

Department of Civil Engineering

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ABSTRACT

Keyword: *Magnification factors (MF); Open ground storey (OGS); stepped Irregular buildings, Seismostruct; Fragility analysis, Reliability analysis, Peak Ground Acceleration (PGA), Performance levels*.

The area of vertically irregular type of building is now having a lot of interest in seismic research field. . Many structures are designed with vertical irregularity for architectural views. Vertical irregularity arises in the buildings due to the significant change in stiffness and strength. Open ground storey (OGS) is an example of an extreme case of vertically irregularity. The typical OGS and stepped types of irregularities are considered in the present study. For OGS buildings, the Magnification factors (MF) are suggested by the design codes, for the design of the open ground storey columns. The present study focus on the performance of typical OGS buildings designed considering various magnification factors as well as the stepped type buildings with different geometry configurations using fragility analysis and reliability analysis. The critical inter-storey drift is considered as an intensity measure.

OGS Building frames designed with various MFs and stepped irregular frames with different infill configurations, and having heights (6, 8 &10 stories) are considered for the present study. Fragility curves are developed for each type of buildings as per the methodology introduced by Cornell (2002). PSDM models are developed for each frames and the corresponding fragility curves are generated. Conclusions on the relative performances of each frame are drawn from the PSDM models and fragility curves. It is

observed that in terms of performance, a building with infill walls in all stories is equally comparable with an OGS framed building with MF of about 1.5. Performance of the OGS frame increases with the increase in MF, but it makes the adjacent storey vulnerable.

The study is extended to the seismic reliability of typical OGS building with various MFs and also the stepped type buildings with different infill configurations in Manipur region (Ukhraul), which is one of the most vulnerable regions in India. The reliability is found out by combining a fragility curve with a seismic hazard curve of the region. The seismic hazard curve for the present study is chosen from the study conducted by Pallav et. al (2012). The reliability of all the frames is evaluated for an earthquake intensity of 2% probability of occurrence of in 50 years at collapse prevention performance level. The performance of the buildings is assessed by comparing the reliabilities achieved with the target reliabilities suggested as per ISO 2394 (1998). It is observed that the frames without any infill walls failed to achieve the target reliabilities. The building provided with infill walls throughout all stories uniformly, achieves the target reliabilities. The stiffness of infill walls is a significant factor that improves the performance of buildings during earthquakes.

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ABBREVIATIONS

OGS	Open Ground Storey
PGA	Peak Ground Acceleration
M	Magnitude
RC	Reinforced Concrete
IS	Indian Standard
MF	Multiplication Factor
NDA	Nonlinear Dynamic Analysis
RSA	Response Spectrum Analysis
SDOF	Single Degree of Freedom
FEMA	Federal Emergency Management Agency
ST1	Stepped type building without infill walls having single step irregularity
ST2	Stepped type building without infill wall having double step irregularity
STFF1	Stepped type building with infill type having single step Irregularity
STFF2	Stepped type building with infill type having double step Irregularity

NOTATIONS

a & b	Regression constant
σ	Standard deviation
μ	Mean value
DL LL & EL	Response quantity due to dead load, live load and earthquake load respectively.
V_B	Design base shear
BM	Bending moment
W	Seismic weight of building
w	Width of the equivalent strut
f_{ck}	Characteristic strength of concrete
f_{co}	Unconfined compressive strength of concrete
COV	Coefficient of variation
CP	Collapse prevention
H	Building height in metres
f_m	Compressive strength of the masonry infill wall
I	Importance factor
Ic	Moment of inertia of the column
IO	Immediate occupancy
f_y	yield strength of reinforcement bar
LS	Life safety
R	Response reduction factor
Z	Zone factor
g	Acceleration due to gravity
I	Importance factor

CHAPTER-1

BACK GROUND AND MOTIVATION

1.1 GENERAL

Vertical irregularities in buildings are very common feature in Urban area. In most of situations, buildings become vertically irregular at the planning stage itself due to some architectural and functional reasons. This type of buildings demonstrated more vulnerability in the past earthquakes. The topics related to of vertical irregularities have been in focus of research for a long time. Many studies have been conducted in this area in deterministic domain. Hence the focus of present study is to assess the relative performances of typical vertically irregular buildings in a Probabilistic domain.

This type of irregularities arises due to sudden reduction of stiffness or strength in a particular storey. For high seismic zone area, irregularity in building is perhaps a great challenge to a good structural engineer. A large number of vertical irregular buildings exist in modern urban infrastructures. Among them Open ground storey as well as stepped types of buildings are very common in Urban India. A typical Open Ground Storey and a Stepped irregular framed building are shown in Figure 1.1.

(a) (b)

Figure 1.1 Vertically irregular buildings. (a) OGS Building (b) Stepped type building.

Open ground storey buildings are also called ‘open first storey buildings’ or ‘pilotis’ ‘stilted buildings’. Because of the scarcity of land, the ground storey is kept open for parking purpose and no infill walls are provided in ground storey but the all above storey are as provided with infill walls

The 2001 Bhuj earthquake (Magnitude M7.9 and PGA 0.11g) was one of the most devastating one to witness the collapse of many open ground storey RC buildings. A typical open ground storey residential building at Ahmadabad. The ground storey columns are badly damaged as shown in Figure1.2 (a) & (b). Figure 1.3 shows the failur of the first storey columns due to shear in Earthquake.

(a)

(b)

Figure 1.2 Failure of the OGS buildings in Bhuj Earthquake (Ref: www.nicee.org)

Figure 1.3 Shear failures of ground storey columns (Ref. **World Housing Encyclopedia, EERI & AIEE)**

Figure1.4 represent the soft storey ground floor with soft storeyed ground floor at China Earthquake of a six storeyed building due to the plastic hinge formation at the ground storey column that tends to increase the inter storey drift at ground floor.

Figure 1.4 Plastic hinged formation at column ends of the ground storey columns in China Earthquake 2008

1.2 CRITERIA FOR VERTCAL IRREGULARUTY IN BUILDINGS

In the previous code of IS 1893, there was no design recommendations particularly for OGS frames mentioned for vertical irregularity. However in the aftermath of Bhuj earthquake was revised in 2002. In recent version of code IS 1893 (2002) (part1)-, incorporated an new design recommendation for OGS buildings.

Clause7.10.3 (a) states “The columns and beams of the soft storey are to be designed for 2.5 times the storey shear and moments calculated under seismic load of bare frame type of buildings”. The magnification factor (MF) 2.5 is examined by Subramanian (2004), Kanitkar and Kanitkar (2001) and Kaushik (2006). The Magnification factor MF 2.5 is not advisable for design of beam as that likely to result a “strong beam – weak column” condition.

It needn’t to design the beams of the soft-storey also to design for higher storey shears as recommended by the above clause. Strengthening of beams will further increase the demand on the columns, and deny the plastic formation in the beams. These recommendations have met with some resistance in design and construction practice due to congestion of heavy reinforcement in the column.

As per IS 1893 (2002) code, five types of irregularities for buildings are listed out as follows:

i) a) **Stiffness Irregularity** - Soft Story: is defined to exist when there is a story in which the lateral stiffness is less than 70% of that in the story above or less than 80% of the average stiffness of the three stories above.

b) **Stiffness Irregularity** - Extreme Soft Story is defined to exist where there is a story in which the lateral stiffness is less than 60% of that in the story above or less than 70% of the average stiffness of the three stories above.

ii) **Weight (Mass) Irregularity** - It is considered to exist where the effective mass of any story is more than 150% of the effective mass of an adjacent story.

iii) **Vertical geometric irregularity -** It shall be considered to exist where the horizontal dimension of the lateral force- resisting system in any story is more than 130% of that in an adjacent story.

iv) **In-plane Discontinuity -** In Vertical Lateral-Force-Resisting Elements is defined to exist where an in-plane offset of the lateral-force-resisting elements is greater than the length of those elements or where there is a reduction in stiffness of the resisting element in the story below.

v) **Discontinuity in Capacity** - The weak story is one in which the story lateral strength is less than 80% of that in the above story. The story lateral strength is the total lateral strength of all seismic-resisting elements sharing the story shear in the consideration direction.

1.3 STEPPED BUILDINGS

Reduction of lateral dimension of the building along their height is categorized as "stepped building". Because of the functional and aesthetic architecture these types of buildings are preferred in modern multi-storeyed building construction. The main advantages of this type of buildings are they provide good ventilation with adequate sun lights to the lower storeys, Sarkar et.al. (2011). this type of building form also provides for compliance with building bye-law restrictions related to `floor area ratio'. Stepped buildings are used to increase the heights of masonry structures by distributing gravity loads produced by building materials such as brick stone etc. These buildings also allow the natural erosion to occur without compromising the structural integrity of the building.

A major earthquake shook cities and villages across the south Asian, several villages in Pakistan and leaving more than 1000 people feared dead. The magnitude 7.6 earthquake killed 157 people across India's Jammu and Kashmir. Scores of people were feared killed or trapped in two 12-storey apartment blocks reduced to rubble in Islamabad as Figure (a) shows.

The stepped building (Timeball Station in Christchurch) at New Zealand is one of the many buildings and landmarks in the city that has been diminished to ruin because of a

6.3 magnitude of earthquake rocked New Zealand that causing widespread damage and killing at least 65 people.(Figure (b).

(a) A stepped type building in Islamabad collapses in Earthquake Ref **@AFP**

(b) New Zealand Earthquake: The country's 'darkest day' **Ref @NYDN**

Figure 1.5 Behaviour of Stepped building in Earthquake

1.4 BARE FRAME BUILDINGS

As per the Indian standard code for earthquake resistance structure designed it is mentioned that while designing a structure the contribution of infill are neglected. The buildings are designed as a bare frame that is the only design of columns and beams are taken into consideration. In the seismic point of view this is the worst case as compared to other building types where the vulnerability is more against lateral loads because of the absence of the infill.

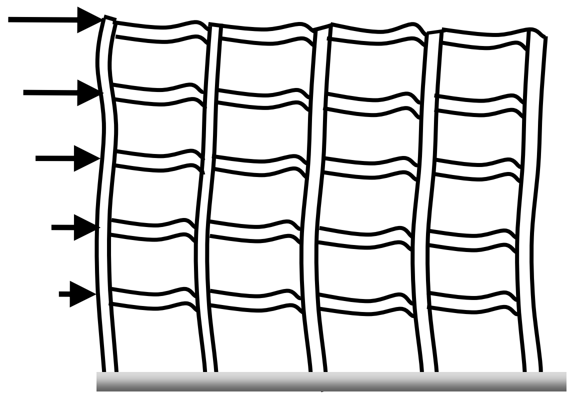


Figure 1.6 Behaviour of Bare frame under lateral load

1.5 TYPICAL INFILLED MASONRY BUILDINGS

The typical infill masonry buildings are the regular buildings considering infill walls provided uniformly through the structures that enhance the strength and stiffness of the structures. The infill walls are considered as a non- structural element from the convenience design practice as per IS code. But in the actual practice the presence of infill walls create a strut compressive action acting diagonally in the direction opposite to the application of the lateral force that may try to counter act the lateral force that causes less deflection. In a bare frame, the resistance to lateral force occurs by the development of bending moments and shear forces in the various beams and columns through the rigid jointed action of the beam-column joints, but in the case of infill frame because of the strut action, contributing to reduced bending moments but increased axial forces in the beams and columns.

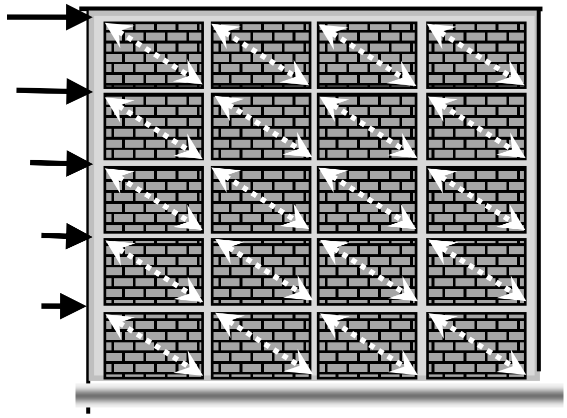


Figure 1.7 Behaviour of fully frame under lateral load

1.6 TYPICAL OPEN GROUND STOREY(OGS) BUILDING

Because of the absence of the infill walls at the ground storey and that of present at all the storey above, the stiffness is sudden decreases which are termed as stiffness irregularity. The base shear is registered by the ground storey columns. Because of the increase in the shear force causes the increase in bending moment and thereby higher curvature that may tends to higher inter storey drift formation at the ground storey and that enhance by the P-Δ effect the plastic hinges are formed. The upper store will move as a single block. This type of collapse is called as soft storey collapse. Because of the decrease in the stiffness at ground storey this type of buildings are considered as the most vulnerable type from the seismic point of view. The fig shows the soft storey collapse of typical OGS buildings.

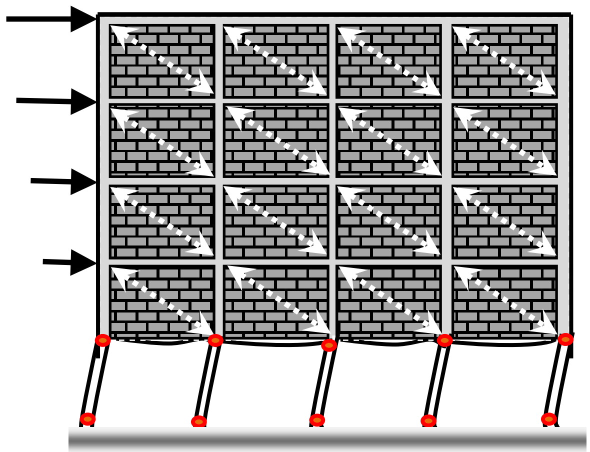


Figure 1.8 Behaviour of OGS frame under lateral load

1.7 OBJECTIVE OF THE STUDY

Based on the previous discussions, the objective of the present study has been identified as follows

✓ To study the seismic performance of buildings with extreme vertical irregularity using fragility analysis.

✓ To develop the Probabilistic seismic demand model for the considered buildings.

✓ To study the relative performance of the building with the regular frames in Probabilistic frame works.

✓ To study the relative performance of OGS buildings designed for various MFs.

✓ To study the seismic hazard analysis of the buildings.

✓ To conduct a reliability analysis and to identify the reliability indices values for all the building frames.

1.8 SCOPE AND LIMITATIONS OF THE STUDY

The RC framed Buildings are considered for the analysis by assuming regular in plan. The buildings considered (6-10 storey buildings) without basement, shear wall and plinth beams. The contribution of Infill walls are considered as non-integral with RC frames. The Out of plane action of masonry walls are neglected in the analysis. The asymmetric arrangement of infill walls are ignored of the buildings. The Soil structure interaction effects are not considered in the analysis. The Flexibility of floor diaphragms are neglected and considered as rigid diaphragm. The base of the column is assumed to be fixed in the analysis.

1.9 ORGANISATION OF THE THESIS

By following the introductory chapter, the next chapter will discuss

The review of the irregularity criteria as mentioned in Indian codes.

- **(i)** Literature review on the various studies conducted on Vertically irregular building in this OGS and stepped types are considered.
- **(ii)** Masonry infill wall models for nonlinear dynamic analysis for the buildings.
- **(iii)** The fragility analysis and the seismic hazard analysis for the buildings are proposed.
- **(iv)** In the chapter3 and 4, a brief description of the outputs as the result and discussions.
- **(v)** Finally, a summary of the present study and the major conclusions are explained in chapter 5.

CHAPTER- 2

REVIEW OF LITERATURE

2.1 INTRODUCTION

The Literature review conducted as part of the present study is divided into two segments. The first part deals with the overview on the fragility analysis of existing design provision of Vertically irregular buildings with regards to the design criteria as per Indian code for various buildings are discussed. In the second part, it based on the seismic hazard analysis and reliability analysis by considering different region in India.

2.2 FRAGILITY ANALYSIS

2.2.1 Vertically Irregular Buildings

Afarani and Nicknam (2012) observed the behaviour of the vertically irregular building under seismic loads by Incremental Dynamic Analysis. They have dealing with eight stories regular building having 2 bays with 4 m width in y direction has and 4 bays with 3 m width in x direction with 3 m storey height is considered. They considered Dead load as 2 ton/m is distributed on beams. To avoid torsional effects they considered symmetric building and steel moment resisting frames which are designed according to IBC 2006 and ANSI/AISC 360-05

Eighteen ground motion records from Pacific Earthquake Engineering Research Centre (PEER) database are collected from Far-Field with distance more than 10 km from site and have Richter magnitudes of 5 to 8 on firm soil. The building is modelled in SeismoStruct-V5 software as a nonlinear dynamic analysis .Steel is modelled as Elastic

Perfectly Plastic (EPP) hysteresis without experience of local and lateral buckling and the connections were failure according to FEMA 440. Maximum inter story drift ratios and first mode spectral acceleration are calculated by Incremental dynamic analysis and IDA curved are plotted to get the collapse points. The analysis of the building is focused on the collapse prevention limit state of the structures. Fragility curves are generated by using Cumulative Distribution Function through the lognormal distribution through collapse points.

The fragility analysis for an irregular RC building under bidirectional earthquake loading has studied by Jeong and Elnashai (2006). For the consideration of the irregularities in structure, the torsion and bidirectional response are utilized as 3D structural response features to represent the damage states of the building irregularities is presented through a reference derivation. A three story RC frame is taken with asymmetric in plan with thickness of slab is 150 mm and beam depth is 500 mm to study the damage assessments. The sectional dimension of C6 is 750×250 mm whereas all other columns are 250×250 mm. Fragility curves are generated by calculating the damage measure with spatial (3D) damage index by statistical manipulation methods and lognormal distributions for response variables Earthquake records consist are of two orthogonal components (Longitudinal and Transverse) of horizontal accelerations and are modified from the natural records to be compatible with a smooth code spectrum. PGAs are taken from a range of 0.05 to 0.4g with a step of 0.05g. For accurate damage assessment of buildings is exhibiting torsion, Planar decomposition method is used where the building is decomposed into planar frame and analysed. The parameters such as top displacement, inter-story drift or a damage index are found out from numerical simulations results. The total damage index is calculated for the planar frames from the backbone envelope curve as a combination of damage due to in-plane monotonic displacement and strength reduction. Coefficient of variation (COV) is found be the ratio of standard deviation to mean value of damage index.

2.2.2 RC Frame Buildings

Tantala and Deodatis (2002) considered a 25 story of reinforced concrete moment resisting frame Building having three-bays. They have generated fragility curves for a wide range of ground motion intensities. They have used time histories are modelled by stochastic processes. Simulation is done by power spectrum probability and duration of earthquake by conducting 1000 simulation for each parameter. The nonlinear analysis is done by considering the P-Δ effects and by ignoring soil-structure interaction. They have considered the nonlinearity in material properties in model with nonlinear rotational springs a bilinear moment-curvature relationship by considering the stiffness degradation through hysteretic energy dissipation capacity over successive cycles of the hysteresis. They have used Monte Carlo simulation approach for simulation of the ground motion. The simulation for the durations of strong ground motions is done at 2, 7 and 12 seconds labels to observe the effects. They considered the effects of the assumption of Gaussianity and duration. They have adopted stochastic process for modelling. The analyses were done by using DRAIN-2D as a dynamic analysis with inelastic time histories data. The random material strengths were simulated for every beam and column using Latin Hypercube sampling.

Murat and Zekeria (2006) studied the yielding and collapse behaviour of RC frame buildings in Istanbul was analysed through fragility analysis based on numerical simulation. They have studied number of stories of buildings as 3, 5 & 7 storeys designed as per Turkish seismic design code (1975).The fragility curves were constructed with the help of the results of regression analysis. They have examined with 12 artificial ground motions for the analysis. Incremental dynamic analysis (IDA) method is used for estimating structural performance under several ground motions. The Characteristic strength of concrete as 16Mpa and two different type of steel as 220Mpa & 420Mpa are used. The uncertainty due to scatter of material as well as the soil structure interaction was ignored in their design mean value of material strength was taken into consideration which was evaluated experimentally.

Performance limit state: inelastic displacement demand and corresponding deformations for immediate occupancy and collapse prevention are evaluated. From the fragility curves finally they have concluded that for the collapse prevention performance level, a good correlation between spectral displacement limit and the number of stories was observed but the same observation was not valid for the immediate occupancy level.

Rota et al (2010) observed the fragility curves for masonry buildings prototype of a three-storey masonry building located in Benevento (southern Italy) which has constructed in1952 are analysed based on stochastic nonlinear analysis. The parameters are found out by Monte Carlo simulation through a program STAC for the analysis. The building used is made of tuff masonry several experimental tests have carried out by Faella et al. The program TREMURI, a frame-type macro-element global analysis program was developed by Gambarotta and Lagomarsino and further modified by Penna for a nonlinear pushover and time history analyses on masonry Buildings. In this study different sources of uncertainty are involved in the problem, by derivation of the probability distributions of both capacity and demand through 3D nonlinear analyses of entire structure. They have used in-plane cyclic shear-compression tests carried out on specimens made of cement mortar and tuff units. The analysis has been done by considering 4 mechanical damages for the structures. Two of them can be identified from the response of a single masonry pier while the other two are found from the global response of the building. First damage state is identified by the attainment of the yield displacement is δy of the bilinear approximation to the capacity curve of a single masonry pier. The second damage state is identified by the drift corresponding to the first shear cracking of the pier is δS which obtained from the experimental test. The third and fourth damage states have been derived from global pushover curves of the building as the third state is assumed to correspond to the attainment of the maximum shear resistance while the fourth state corresponds to the attainment of 80% of that value. All the mechanical properties of the structure are assumed to be random variables, the mean value and standard deviations are calculated by normal probability distributions of the building typology.

Erberik (2008) studied the low-rise and mid-rise reinforced concrete (RC) buildings through Fragility analysis that characteristics in the Duzce Damage database which effected by two devastating earthquakes in 1999 at Marmara region in turkey. They have considered the buildings of number of stories ranges between two and six. In the analysis the building having two and three stories are regarded as low-rise (LR) and buildings having four to six stories are considered as mid-rise (MR).They have studied with 28 RC buildings extracted from a building database of around 500 buildings in Duzce. Post-earthquake damage assessments of the buildings were available. The Duzce damage database has been used previously by other researchers.100 corrected ground motion records have collected from different parts of the world with a range of magnitude between 5.1 and 7.8 are used for the analysis. The ground motion set is divided into 20 Groups each of five with PGV intervals of 5 cm/s, the buildings are modelled as bare frame or infill frame. In the study they subdivided the building as low-rise bare frame type, low-rise infill frame type, mid-rise bare frame type and mid-rise infill frame type. The low-rise and mid-rise RC structures are analysed as a single degree of freedom system with the global response statistics of simplified (or equivalent) analytical models. They have considered three structural Parameters as period, strength ratio and the post-yield to initial stiffness ratio. First mode parameters are obtained and the capacity spectra are constructed in acceleration–displacement response spectra (ADRS) pattern. Then these capacity spectra are identified by the bilinearization method in FEMA356 and Capacity curves of the structural models were obtained by SAP2000. Sampling is done by size on the fragility functions, structural simulations using LHS technique by using MATLAB. The Building damages were observed in four stages as none, light, moderate and severe or collapsed. The performance limits of building for Serviceability limit state, Collapse prevention limit and Damage control limit state are studied. Finally fragility curves are generated for different classes of RC buildings and compared with the actual field data.

Guneyisi and Altay (2008) observed the behaviour of existing R/C office buildings through fragility curves considering the conditions as before and after retrofitted by fluid viscous (VS) dampers. Braced frames are considered at the middle bay of the frame with passive fluid VS dampers at each brace. A 12-storey office building designed as Turkish seismic design code version (1975) from Istanbul. VS dampers are used for retrofitting, designed as FEMA 273–274 with different effective damping ratios of 10%, 15%, and 20%. Main structural system of the building consists of moment resisting R/C frames in two directions with 12-storey located at moderate seismic zone with relatively stiff soil type as per Turkish seismic design code has taken. The storey height of the building is 2.75 m with 989 m2 floor area. 240 earthquake ground motions are generated by considering the spectral representation methodology based on the stochastic engineering Approach with the help of MATLAB program limited to 1PGA. The R/C building is modelled as a three-dimensional analytical model of the building was established in ETABS version 7.2 Structural Analysis Program for the analysis. For the seismic response of the buildings are focused by the nonlinear time history analyses with push over analysis by IDARC version 6.1 programs. The characteristic strength and yield strength ia considered as of 16 MPa and 220 MPa. The fragility curves are generated for four damage states as slight, moderate, major, and collapse conditions and Load-deformation relationship for the weak axis (y-axis) and the structural damage limit values determined for each type of damage. The fragility curve generated for the building are concluded that with the help of retrofitting the failure chances of building becomes minimized such that the before retrofitting is more fragile than after retrofitting case.

Samoah (2012**)** observed the fragility behaviour of non-ductile reinforced concrete (RC) frame buildings in low - medium seismic areas and they have preferred at Accra which is the capital of Ghana, West Africa. The structural capacity of the buildings is analysed by inelastic pushover analysis and seismic demand is analysed by inelastic time history analyses**.** Then the fragility curves are drawn. They have examined with 3 generic non-ductile RC frame buildings having symmetrical and regular in both plan and elevation are designed according to BS 8110 (1985). The buildings taken into consideration are a 3-storey and 3-bay, a 4-storey and 2-bay and a 6-storey and 3 bay buildings to get an appropriate result. The structure was modelled using 35 and 60% of the gross sectional areas of beams and columns. A macro-element program IDARC2D (1996) was developed as the inelastic static and dynamic analysis of non-ductile RC frames. The analysis for the non-ductile RC frame buildings, modelling are done adequately based on their structural properties.

Rajeev and Tesfamariam (2012) studied the seismic performance of non-code conforming RC buildings designed for gravity loads. The analysis highlights the need for reliable vulnerability assessment and retrofitting. The vulnerability is compounded since the RC buildings are subject to different irregularities such as weak storey, soft storey, plan irregularities sand other types Fragility based seismic vulnerability of structures with consideration of soft storey(SS) and quality of construction(CQ) is demonstrated on three-, five-, and nine-storey RC frames designed prior to 1970s. Probabilistic seismic demand model (PSDM) for considered structures is developed, by using the nonlinear finite element analysis. Further, the fragility curves are developed for the three structures considering SS, CQ and of their interactions. Finally, confidence bounds on the fragilities are also presented as a measure of their accuracy for risk-informed decision-making. With the PSDM models the corresponding fragility curves are generated. in the analysis. They concluded that the vertical irregularities and construction quality in seismic risk assessment have a significant influence in the decision making phase. The proposed approach of developing a predictive tool can enhance regional damage assessment tool, such as HAZUS, to develop enhanced fragility curves for known SS and CQ.

2.2.3 OGS Buildings

Davis and Menon (2004) examined the presence of masonry infill panels modifies the structural force distribution significantly in an OGS building. They considered verities of building case studies by increasing the storey heights and bays in OGS buildings to study the change in the behaviour of the performance of the buildings with the increase in the number of storey and bays as well as the storey heights. They observed that with the total storey shear force increases as the stiffness of the building increases in the presence of masonry infill at the upper floor of the building. Also, the bending moments in the ground floor columns increase and the failure is formed due to soft storey mechanism that is the formation of hinges in ground storey columns.

Scarlet (1997) identified the qualification of seismic forces of OGS buildings. A multiplication factor for base shear for OGS building was proposed. The modelling the stiffness of the infill walls in the analysis was focused. The effect of in Multiplication factor with the increase in storey height was studied. He observed the multiplication factor ranging from 1.86 to 3.28 as the number of storey increases from six to twenty.

Hashmi and Madan (2008) conducted non-linear time history and pushover analysis of OGS buildings. They concluded that the MF prescribed by IS 1893 2002 for such buildings is adequate for preventing collapse.

Sahoo (2008) observed the behaviour of open-ground-storey of Reinforced concrete (RC) framed buildings having masonry at above storey by using Steel-Caging and Aluminum Shear-Yielding Dampers. He has introduced a simple spring-mass model for the design of braces for adequate strength and stiffness requirements of the strengthening system. He has taken a 5 storey 4 bay non-ductile RC frame having open ground- storey for his observation. And also reduced scale 1storey 1 bay RC frame was analysed experimentally under constant gravity loads and reversed cyclic gradually increasing lateral Displacement by Full strengthening technique. For flexural strength and inelastic rotation at a target yield mechanism the performance-based design method was developed to withstand the probable seismic demand as the lateral strength, inelastic deformation and energy dissipation demand on structures. He observed for load transferring assemblies the steel cage-to-RC footing connection and brace-to-steel cage connection exhibited excellent performance under lateral cyclic loading without any sign of premature failures. Whereas the RC frame strengthened with only steel caging exhibited the improved lateral strength, drift capacity and energy dissipation potential as compared to the non-ductile frame but could not avoid collapse completetely.

Patel (2012) proposed both linear as the Equivalent Static Analysis and Response Spectrum Analysis and the nonlinear analyses as the Pushover Analysis and Time History Analysis for Low-rise open ground storey framed building with infill wall stiffness as an equivalent diagonal strut model. The effect of the infill wall is studied considering the Indian standard code IS 1893 2002 criteria mention for OGS buildings. She observed that the analysis results shows that a MF of 2.5 is too high to be multiplied to the beam and column forces of the ground storey of the buildings. Their study conclude that the problem of open ground storey buildings cannot be identified properly through elastic analysis as the stiffness of open ground storey building and a similar bare-frame building are almost same.

2.2.4 Stepped Buildings

Sarkar et al (2009) considered the irregularity in stepped framed building by considering Regularity index.78 building frames with uniform number and bay width of 4 and 6m respectively with varying degree of stepped irregularity are considered seven numbers of buildings with different height are also included without considering step.50 modes are focused for four different cases of building. They observed by histogram that with the increases in irregularity, the first-mode participation decreases with increased participation on some higher modes. Delhi Secretariat building ten-storied office building located in New Delhi (Seismic Zone IV with designed PGA of 0:24g as per IS 1893:2002).The modelling and analysis were done by using a program SAP2000.

Kim & Shinozuka (2004) studied the fragility analysis of two sample bridges retrofitted by steel jacketing of bridge columns in southern California. Among the two bridges the first one bridge was 34m long with three span with two half shells of rolled steel plate and a RC deck slab 10m width supported by 2pairs of circular columns(each having 3 columns of diameter as 0.8 m) with abutments. And the second bridge was 242m long with a deck slab dimension (13m wide &2m deep) which supported by 4 circular columns of 2.4m diameter and height of 21m have an expansion joint at centre was taken.60 ground acceleration time histories were collected from the Los Angeles the historical records and then Adjusted. After that then they have categorized into 3groups each of having 20data.The bridges were modelled as a two-dimensional response analysis with a computer program SAP2000 or nonlinear finite method. The fragility curves were developed by considering before and after column retrofit with steel jackets cases with probabilities of exceedence of 10% in 50 years, 2% in 50 years and 50% in50 years, respectively. Nonlinear response characteristics associated with the bridges are based on moment–curvature curve analysis. They considered two-parameter lognormal distribution functions by the median and log standard deviation to analysis the fragility curves. They have done the analysis for different performance levels as no cracking, Slight Cracking Moderate, Extensive Incipient column collapse Complete. The fragility curves were generated from the experimental outputs and then compared.

Zentner et al (2008) observed the seismic probabilistic risk assessment (PRA) for seismic risk evaluation of nuclear plants is studied through fragility analysis in the analysis. They considered coupled model consisting of a supporting structure that is containment building modelled as 3D stick model and also the secondary system that represent a reactor coolant system which is modelled as a beam elements consists of a reactor vessel and four loops having steam generator. Primary pump and piping in each loop is multi-supported by 36 supports. Four upper lateral supports placed at the top of each steam generator and three lower lateral supports for guidance and safety of steam generator & reactor vessel. Statical estimation of parameters through fragility curves for a nuclear power plant was studied by means of numerical simulation.. They have generated 50 artificial ground motions time histories and analysed as a nonlinear dynamic response of the site response spectrum for a rocky site. The ground motions are modelled by stochastic process from artificial time histories data. All the numerical computations they have carried out using Code Aster open source FE-software for the output results. They have considered two configurations in the analysis. First they have considered the uncertainties related to soil and earthquake in the analysis and then they considered the uncertainties due to earthquake ground motion as well as structural and mode in the plant equipment.

Ozel and Guneyisi (2011) studied a mid-rise RC frame building retrofitted with eccentric steel brace was observed through Fragility analysis. A six storey RC frame building, designed as per Turkish seismic design code 1975 located in a high-seismicity region of Turkey was taken in the study. In building typical beam and column was considered without shear wall. The steel braces (K,V&D type) they have used 4different distribution to observe the behaviour. The fragility curves were developed from the inter storey drift by means of nonlinear time history analysis. The fragility curves developed for the original building for different damage levels.200 earthquake data were considered that generated by using MATLAB program. Modelling was done as a 2D analysis by using a software SAP2000 nonlinear version 11.The median and standard deviation of the ground motion indices for each damage level were obtained by performing linear regression analysis for different performance levels. They observed the different damage levels as slight, moderate, major, and collapse for the building. The fragility curves were developed for before and after retrofitting with steel braces. They concluded after retrofitting with steel braces were less fragile compared to those before retrofit. And the distributions of the eccentric steel braces were slightly affecting the seismic reliability of the braced frames. First distributions (K1, V1, or D1) gave the greatest and fourth distributions (K4, V4, or D4) gave the least seismic reliability.

Marano et al (2011) the fragility curves are developed that based on the classification and structures provided by the Hazus database with the uses of stochastic dynamic analysis. Types for the buildings are taken as 2 storeys and 5 storeys buildings with both low seismic design and medium seismic design are considered. A displacement based damage index is adopted for the fragility analysis. The structure considered is a nonlinear single degree of freedom system (SDOF).Response to seismic action, modelled by means of the modulated Clough and Penzien process, is achieved by using stochastic linearization technique and covariance analysis. Fragility curves are obtained by means of an approximate threshold crossings theory. A sensitivity analysis has been performed with respect to structural parameters and also considering different soil types. From the sensitivity analysis carried out considering structural mechanical parameters it can be deduced that all the parameters affect the fragility curves, except the stiffness ratio α which influences only the fragility curve which corresponds to the heavy damage state.

Cornell et al (2002) investigated a formal probabilistic framework for seismic design and assessment of structures and its application to steel moment-resisting frame buildings based on the 2000 SAC, Federal Emergency Management Agency (FEMA) steel moment frame guidelines. The framework is based on realizing a performance objective expressed as the probability of exceedance for a specified performance level. That related to ‘‘demand’’ and ‘‘capacity’’ of that are described by nonlinear, dynamic displacements of the structure. l of the spectral acceleration at the approximate first. Probabilistic models distributions were used to describe the randomness and uncertainty in the structural demand given the ground motion level, and the structural capacity. A common probabilistic tool the total probability theorem was used to convolve the probability distributions for demand, capacity, and ground motion intensity hazard. This provided an analytical expression for the probability of exceeding the performance level as the primary product of development of framework. Consideration of uncertainty in the probabilistic modelling of demand and capacity allowed for the definition of confidence statements for the likelihood performance objective being achieved

2.3 SEISMIC HAZARD AND RELIABILITY ANALYSIS

Pallav et al (2012) estimated the spectral acceleration of the Manipur region through the probabilistic seismic hazard analysis (PSHA). The area considered for the analysis is divided into different zones. By consideration of past earthquake data the earthquake recurrence relations are evaluated for the analysis Seenapati, tamenglong, churachandpur, chandel, imphal east, Imphal west, Ukhrul, Thoubal and Bishnupur places belongs to that region are considered for the analysis. Counter maps are considered for the different places of Manipur region by considering the variation of peak ground acceleration for return periods. These results may be of use to planners and engineers for selection of site, earthquake resistant structures designing and, may help the state administration in seismic hazard mitigation.

Ellingwood (2001) estimated the earthquake risk assessment of the building by applying the probabilistic risk analysis tools for two decades. He focused on the3 probability based codified designed and reliability based condition asse3ssment of existing structures. The steel frames weld connected are designed. A nonlinear dynamic analysis is done to study the behaviour in the importance of inherent randomness and modelling uncertainties in the performance of the buildings through fragility analysis. The seismic hazard analysis is done by considering the ground motion from California strong ground motion network.

Dymiotis et al (2012) studied on the probabilistic assessment of reinforced concrete frames infilled with clay brick walls and subjected to earthquake loading. The adopted methodology extends that previously developed by the writers for bare RC frames designed with EC8 by introducing additional random variables to account for the uncertainty in the masonry properties. Masonry infill walls are modelled as a four-nodded isoparametric shear panel elements of complex hysteretic behaviour. Dynamic inelastic time-history analyses of 2D frame models are carried out using DRAIN-2D/90. The program utilizes the lumped mass approach and point hinge idealizations for line members.

Quantification of the latter is achieved through the use of experimental data describing the difference in force-displacement behaviour between bare and infill frames. The vulnerability and seismic reliability of two 10-story, three-bay infill frames (a fully infill one and one with a soft ground story) are derived and subsequently compared with values corresponding to the bare frame counterpart. The seismic vulnerability is found out for two limit state levels as serviceability and ultimate limit state They concluded that failure probabilities, at the ultimate limit state, are highly sensitive to the structural stiffness; hence, bare frames benefit from lower spectral ordinates than infill ones. Nonetheless, all structural systems studied appear to be exposed to a reasonably low seismic risk.

Celik and Ellingwood (2010) observe the seismic performance of the reinforced concrete frames belongs to low seismic region are designed and analysed for gravity loads. They considered the uncertainty in the material properties and structural systems (i.e. structural damping, concrete strength, and cracking strain in beam–column joints) have the greatest impact on the fragilities of such frames. Confidence bounds on the fragilities are also presented as a measure of their accuracy for risk-informed decision-making, for prioritizing risk mitigation efforts in regions of low-to-moderate seismicity.

Bhattacharya et al (2001) focused on the development of the target reliability of the novel structures that calibrated to existing structures. They adopted a general risk methodology of reliability framework is considered for finding out the significant limit state and the identification of the target reliability for the structures analytically. The methodology is illustrated with the US Navy's Mobile Offshore Base concept is the unique offshore structure in terms of function and size, and where no industry standard exists. A survey of reliability levels in existing design standards and engineered structures, target reliabilities recommended by experts, and analytical models for establishing acceptable failure probabilities is presented. The MOB target reliabilities presented here are subject to modification in the actual acquisition phase when more input becomes available. It is concluded that setting target reliabilities for high-value novel structures is not an engineering decision alone active involvement on the part of the owners and policy-makers is also required.

Sykora, M., & Holicky, M., (2011) investigated the same target reliability level for the assessment of existing structures. The variation of the cost as well as the reliability index is determined by considering the different parameters. By considering the various codes the target reliability has estimated for the building and based upon this the performance levels are evaluated. The target reliability levels are primarily dependent on the failure consequences and on the marginal cost per unit of a decision parameter; upgrade costs independent of the decision parameter; remaining working life and discount ratio are less significant. The design values are specified on the basis of an appropriate reliability index (β).

2.4 SUMMARY

Overviews of guidelines for vertically irregular buildings are carried out in the first chapter. The review of the study indicates that there are numerous research efforts found on the seismic behaviour of RC buildings, OGS buildings and on the modelling infill walls for linear and nonlinear analysis. Also with regard to seismic performance of the vertically irregular buildings, there are few studies conducted. But all this studies are based on a deterministic approach. The main motivation is to study the performance of the vertically irregular buildings and to fine tune the design guidelines as per the Indian standards. For example, with regard to an OGS building, the IS 1893(2002) suggests a multiplication factor of 2.5 for ground storey columns. The multiplication factor proposed by IS 1893 (2002) needs to be more of rational than an empirical number. The first part the present study will attempt to propose the multiplication factors for performance objectives of the OGS buildings. In the second part as seismic hazard analysis and reliability analysis, there are very few literatures are found based on the structures. The seismic hazard analysis is adopted for the OGS buildings and the stepped type buildings by considering the criteria from various codes by identifying the reliability index calculation for the buildings to evaluate the appropriate MF values for the design of the buildings belongs to various region of India.

CHAPTER-3

PERFORMANCE ASSESSMENT OF VERTICALLY IRREGULAR BUILDINGS USING FRAGILITY CURVES

3.1 GENERAL

This chapter focused on the back ground information regarding the formulation and the methods used for the development of the fragility curves. The fragility analysis has done by regression analysis that influenced by the seismic intensity measure and the structural demand. For the study the seismic intensity measure is considered as the ground motion and the structural demand is the engineering demand parameter which is the inter storey drift capacities in terms of peak ground acceleration for generation of fragility curves for different performance levels.

3.2 DEVELOPMENT OF FRAGILITY CURVES

A fragility function represents the probability of exceedance of the selected Engineering Demand Parameter (EDP) for a selected structural limit state (DS) for a specific ground motion intensity measure (IM). These curves are cumulative probability distributions that indicate the probability that a component/system will be damaged to a given damage state or a more severe one, as a function of a particular demand.Fragility curve damaged to a given damage state or a more severe one, as a function of a particular demand.Fragility curve can be obtained for each damage state and can be expressed in closed form as using Eq.3.1

$$P\,(C\text{-}D \leq 0|IM) = \Phi\left(\frac{\ln\frac{S_d}{S_c}}{\sqrt{\beta_{d|IM}^2+\beta_c^2}}\right) \tag{3.1}$$

Where, C is the drift capacity, D is the drift demand, S_d is the median of the demand and S_c is the median of the chosen damage state (DS). $\beta_{d/IM}$ and β_c are dispersion in the intensity measure and capacities respectively. Eq. 3.1 can be rewritten as Eq. 3.2 for component fragilities (Nielson, 2005) as,

$$P\,(DS|IM) = \Phi\left(\frac{\ln IM - \ln IM_m}{\beta_{comp}}\right) \tag{3.2}$$

Where, IMm = $\exp\left(\frac{\ln S_c - \ln a}{b}\right)$, *a* and *b* are the regression coefficients of the probabilistic Seismic Demand Model (PSDM) and the dispersion component, β_{comp} is given as,

$$\beta_{comp} = \sqrt{\frac{\beta_{d|IM}^2+\beta_c^2}{b}} \tag{3.3}$$

The dispersion in capacity, βc is dependent on the building type and construction quality. For βc, ATC 58 50% draft suggests 0.10, 0.25 and 0.40 depending on the quality of construction. In this study, dispersion in capacity has been assumed as 0.25.

It has been suggested by Cornell et. al (2002) that the estimate of the median engineering demand parameter (EDP) can be represented by a power law model, which is called a Probabilistic Seismic Demand Model (PSDM) as given in Eq. 3.4.

$$\widetilde{EDP} = a(IM)^b \tag{3.4}$$

In this study, inter-storey drift (δ) at the first floor level (ground storey drift) is taken as the engineering damage parameter (EDP) and peak ground acceleration (PGA) as the intensity measure (IM).

3.3 GROUND MOTION DATA

The number of ground motions required for an unbiased estimate of the structural response is 3 or 7 as per ASCE 7-05. However, ATC 58 50% draft recommends a suite of 11 pairs of ground motions for a reliable estimate of the response quantities. ASCE/SEI 41 (2005) suggests 30 recorded ground motions to meet the spectral matching criteria for NPP infrastructures. A set of thirty Far-Field natural Ground Motions are collected from Haselton and Deierlein (2007). These are converted to match with IS 1893 (2002)) spectrum using a program, WavGen developed by Mukherjee and Gupta (2002). Figure 3.1 shows the Response spectrum for converted ground motions along with Indian spectrum.

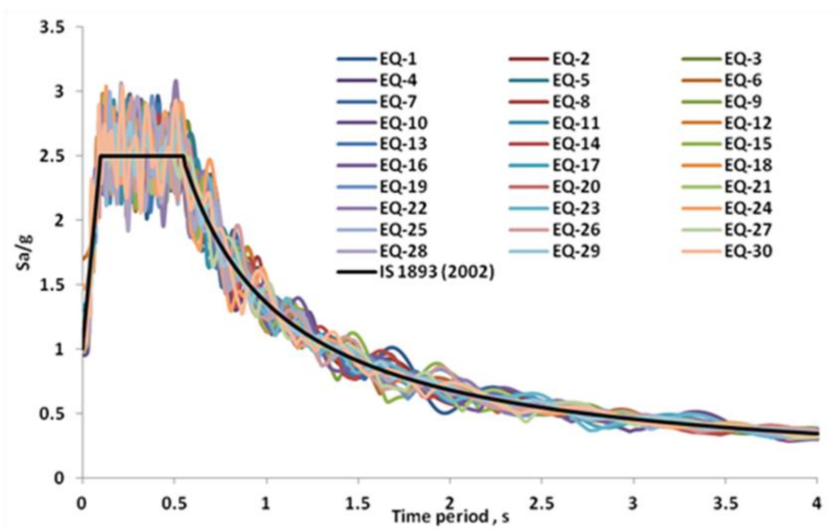
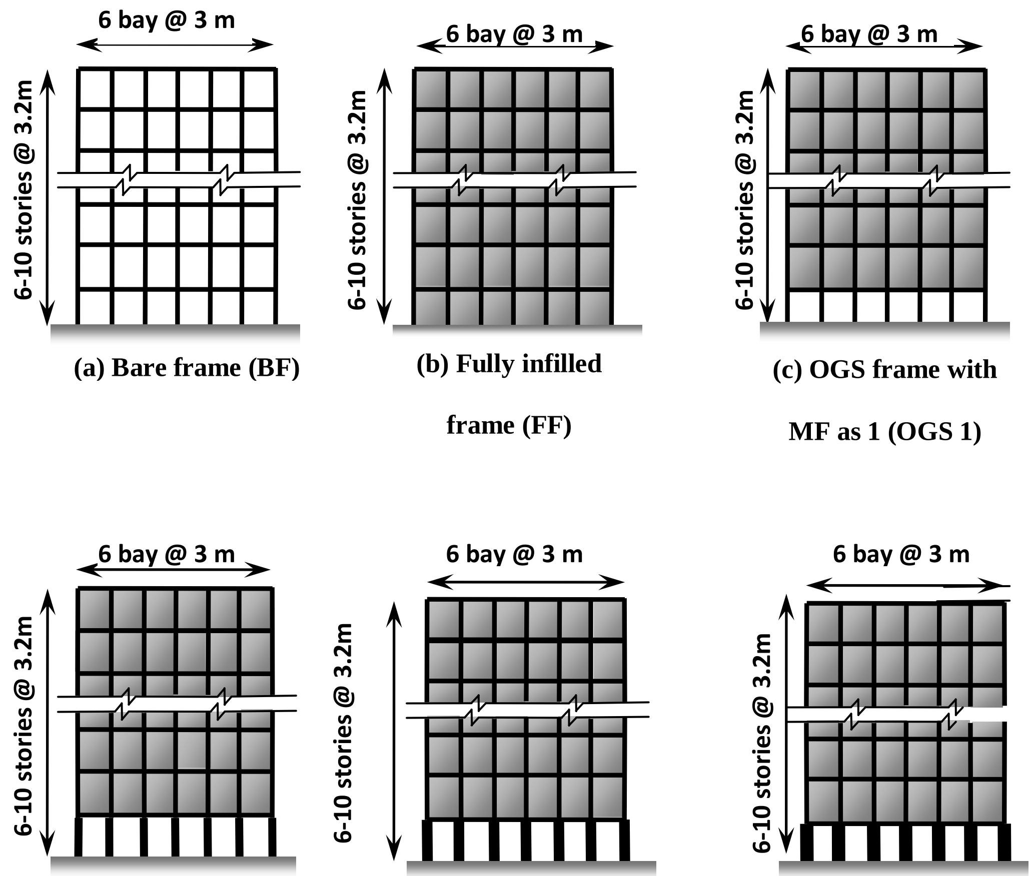


Figure 3.1 Response Spectra for 30 converted ground motions along with IS 1893 (2002) design spectrum

3.4 BUILDING DESIGN

3.4.1 Open Ground Storey building frames with different Multiplication Factors

The buildings frames considered for numerical analysis in the present study are located in Indian seismic zone V with medium soil conditions. These frames are designed as an Ordinary moment resisting frames, seismic loads are estimated as per IS 1893 (2002) and the design of the RC elements are carried out as per IS 456 (2000) standards. The characteristic strength of concrete and steel were taken as 25MPa and 415MPa. The buildings are assumed to be symmetric in plan. Typical bay width and column height in this study are selected as 3m and 3.2m respectively for all the frames. The different building configurations are chosen from 6 storeys to 10 storeys by keeping the number of bays as six for all the frames. The building configurations for the OGS building with different MF of different frames are shown in Figure 3.2. The sectional details of the ground storey columns obtained for various MFs are provided in Table 3.1.



(a) Bare frame (BF)

(b) Fully infilled frame (FF)

(c) OGS frame with MF as 1 (OGS 1)

(d) OGS frame with MF as 1.5 (OGS 1.5)

(e) OGS frame with MF as 2 (OGS 2)

(f) OGS frame with MF as 2.5 (OGS 2.5)

Figure 3.2 Elevation of building frames considered

The different building configurations are chosen from 6 storeys to 10 storeys by keeping the number of bays as six for all the frames. The building configurations of different frames are shown in Figure 3.2. The sectional details of the ground storey columns obtained for various MFs are provided in Table 3.1. Explain the stepped configurations nicely.

Table 3.1 Details of Open Ground Storey frames

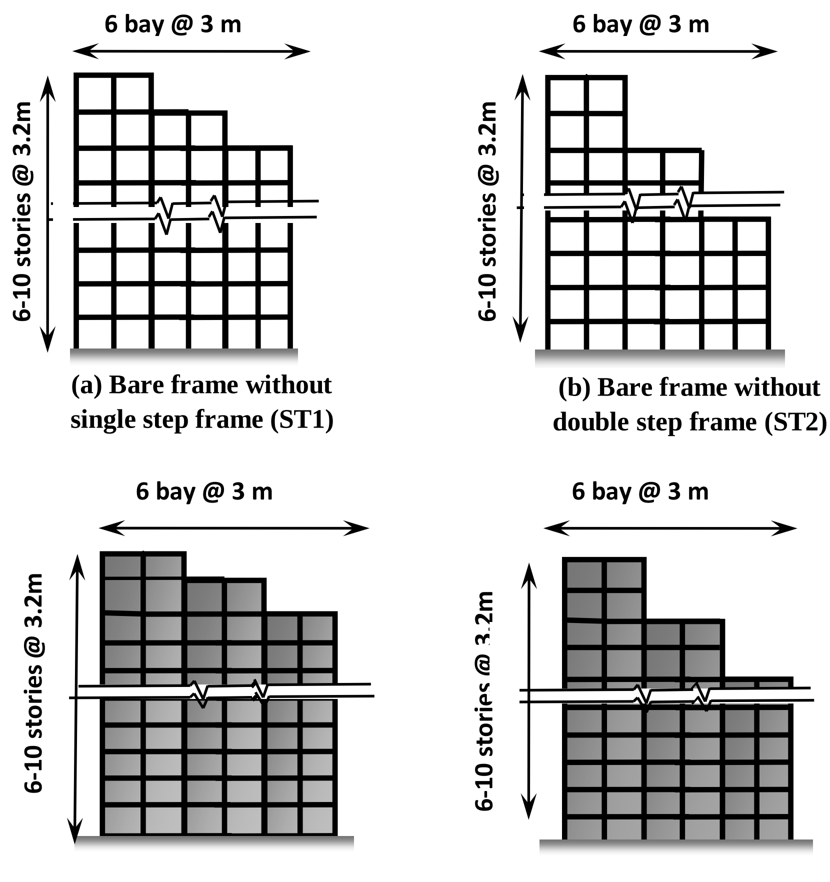
Sl No.	**Frame designation**	**Designation**	**Ground storey column section**
1	6 to 10stories and 6 bays, Full Frame	FF	350 x 350
2	6 to10 stories and 6 bays, OGS (M.F =1)	OGS 1	350 x 350
3	6 to 10 stories and 6 bays, OGS (M.F =1.5)	OGS 1.5	450 x 450
4	6 to10 stories and 6 bays, OGS (M.F =2)	OGS 2	600 x 600
5	6 to 10 stories and 6 bays, OGS (M.F =2.5)	OGS 2.5	750 x 750

3.4.2 Building frame with stepped irregularities

The buildings frames with vertically irregular frames are considered for performance assessment using fragility curves. The buildings frames are assumed to be located in Indian seismic zone V with medium soil conditions. These frames are designed as an Ordinary moment resisting frames, seismic loads are estimated as per IS 1893 (2002) and the design of the RC elements are carried out as per IS 456 (2000) standards. The characteristic strength of concrete and steel were taken as 25MPa and 415MPa. The buildings are assumed to be symmetric in plan. Typical bay width and column height in this study are selected as 3m and 3.2m respectively for all the frames. Table 3.2 presents the description and designation of the vertically irregular frames considered. The elevations of all the vertically irregular frames are displayed in Figures 3.3a to 3.3d. ST1 stands for vertically irregular frame with single storey steps without any infill walls. STFF1 represents vertical irregular buildings with single storey steps with infill walls uniformly throughout.

Table3.2 Details of stepped irregular frames

Sl No.	**Frame Description**	**Designation**
1	6 to 10 storey and 6 bay, BARE framed with single storey stepped type	ST1
2	6 to 10 storey and 6 bay, BARE framed with double storey stepped type	ST2
3	6 to 10 storey and 6 bay, FF framed with single storey stepped with infill type	STFF1
4	6 to 10 storey and 6 bay, FF framed with double storey stepped with infill type	STFF2



(a) Bare frame without single step frame (ST1)

(b) Bare frame without double step frame (ST2)

(c) Bare frame with single step frame (STFF1)

(d) Bare frame without double step frame (STFF2)

Figure 3.3 Elevation of stepped building considered

3.5 SAMPLING

Material properties of concrete, steel and masonry used in the construction are random in nature. To incorporate the uncertainties in concrete, steel and masonry strength, a Latin Hypercube sampling scheme is adopted using MATLAB (2009) program. Table 3.3 shows the mean and covariance of each random variable considered. The values for concrete and steel are taken from Ranganathan (1999) and that for masonry is taken from Kaushik et.al. (2007).

Table 3.3 Details of random variables used in LHS scheme

Material	Variable	Mean	COV(%)	Distribution	Remarks
Concrete	f_{ck} (MPa)	30.28	21	Normal	Uncorrelated
Steel	f_y (MPa)	468.90	10	Normal	Uncorrelated
Masonry	f_m (Mpa)	6.60	20	Normal	Uncorrelated

3.6 MODELLING AND ANALYSIS

30 models are considered for each case, which is modelled in Seismostruct (2009) for nonlinear analysis. Concrete is modelled as per Mander et al. (1988) and reinforcements using a bilinear steel model with kinematic Strain hardening. Infilled masonry walls are modelled according to Crisafulli (1997) which takes into account of the stiffness and strength degradations in each cycle, which is implemented in SeismoStruct. Hilber-Hughes Taylor series scheme is adopted for the time step analysis and skyline technique is used for matrix storage.

3.7 PERFORMANCE LEVELS

Performance levels are the levels to indicate the damage states of the building under seismic loading. Performance levels for a typical building pushed laterally to failure is shown in the Figure 3.4& table 3.4 .A typical Three performance levels, Immediate Occupancy (IO), Life safety (LS) and collapse Prevention (CP),are considered in the present study. The inter-storey drift (S_c) corresponding to these performance levels has been taken as 1%, 2% and 4% respectively as per FEMA356.

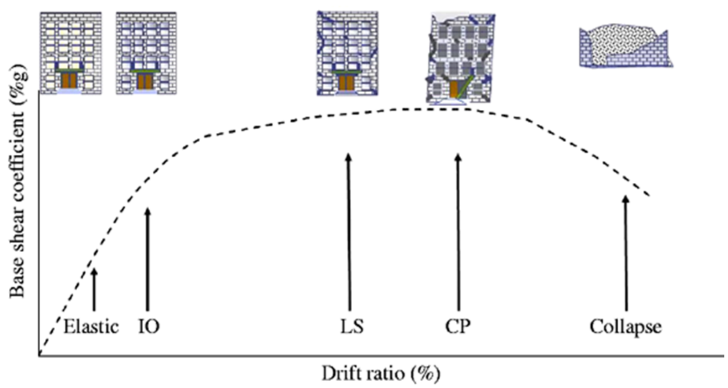


Figure 3.4 Damage states of a typical building pushed to failure (Courtesy, FEMA356)

Table 3.4 Damage limits with various structural performance levels (FEMA356) for RC frames

Limit states designation	**Performance level**	**Inter-storey Drifts Sc for MRF, (%)**
IO	Light repairable damage	1
LS	Moderate repairable damage	2
CP	Near collapse	4

3.8 PERFORMANCE OF 10 STOREY 6 BAY OGS BUILDING FRAMES

3.8.1 PSDM models for Open Ground Storey building frames with different Multiplication Factors

For developing a fragility curve, Nonlinear dynamic analyses of 30 building models are conducted and the maximum inter storey drift (ID) at any storey is recorded. The parameters of the power law model are found out by regression analysis for each frame to develop PSDM model.

The parameters, ‘a’ and ‘b’ of the PSDM models obtained for all the frames are summarised in the Table 3.5. A comparison of PSDM models for 10 storeyed building case study for all the infill wall configurations are drawn in a log-log graph as shown in the Figure 3.5. It can be seen that the inter storey drifts for bare frame is significantly higher than all the remaining cases. This is due to the less lateral stiffness of the bare frame by neglecting infill walls. The inter-storey drift of OGS building designed for MF 1.0 is more than that of regular building (FF), in which brick masonry infill walls are provided in all the storeys uniformly. The maximum inter-storey drift of OGS frame designed with MF of 1.5 is less by about 16 % (maximum) than that for regular frame (FF) for all PGA.

It can be seen that as the MF increases the inter-storey drift decreases. The inter-storey drift of OGS building designed with MF of 2.5 is about 50% less than that in an OGS frame designed using a MF of 2.0. Similarly, the maximum inter-storey drift reduction in an OGS building designed with MF of 2.0 compared to that of MF of 1.5 is about 33%.

The variation of maximum inter-storey drift with the MF used for the design of OGS buildings is plotted in Figure 3.6.

Table 3.5 Parameters of Probabilistic Seismic Demand Models for OGS buildings for 10, 8 and 6 storeyed frames for various infill walls configurations

Building types	**10 Storey 6 Bay**		**8 Storey 6 Bay**		**6 Storey 6 Bay**	
	A	**b**	**a**	**b**	**a**	**b**
BF	100.3	1.019	104.63	1.1085	156.62	1.2108
FF	12.522	1.1166	11.925	1.0964	11.932	1.098
OGS 1	13.975	0.9815	14.065	0.9748	16.921	1.0053
OGS 1.5	10.558	1.0549	11.606	1.0802	13.14	1.0976
OGS 2	7.3815	1.1606	7.7746	1.0908	9.6038	1.2256
OGS 2.5	3.472	1.0853	4.6186	1.1267	6.2698	1.2852

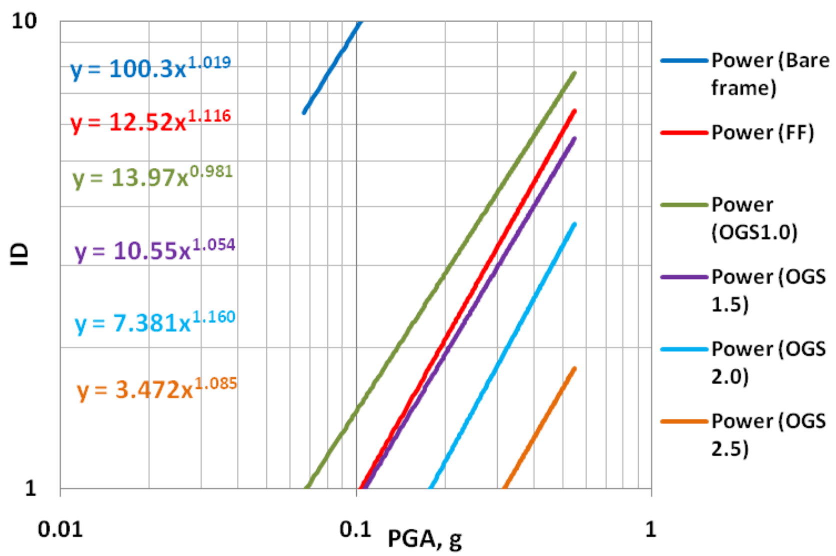


Figure 3.5 Comparison of PSDM models for various OGSframes, Bare and Full Infilled Frame

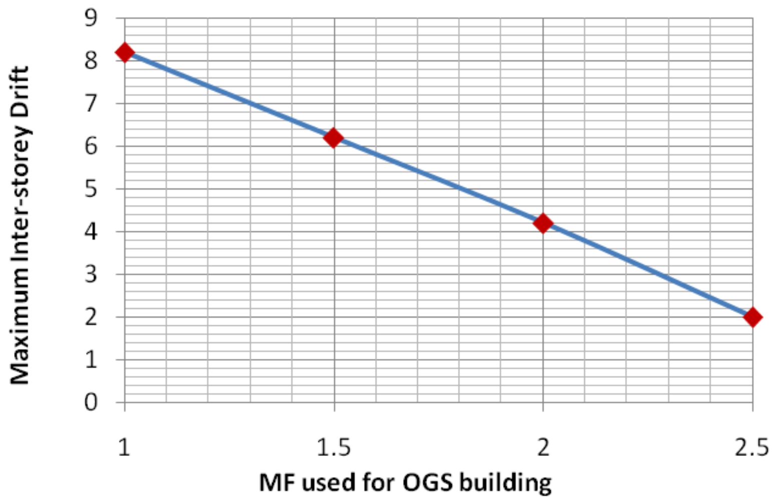
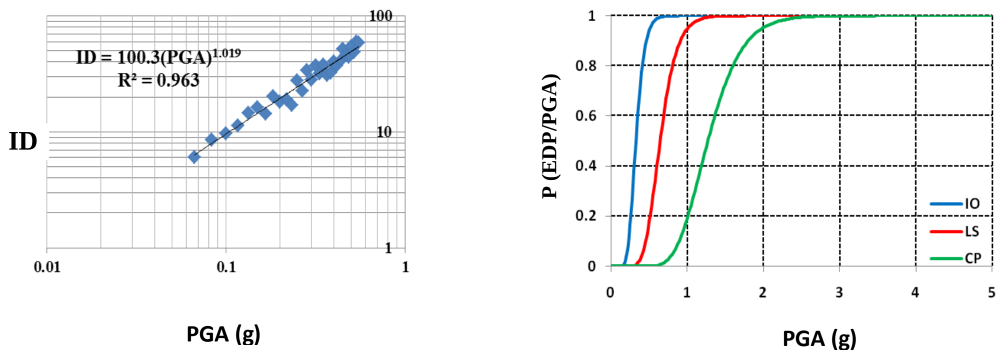


Figure 3.6 Variation of Maximum inter-storey drift with MF used for OGS building

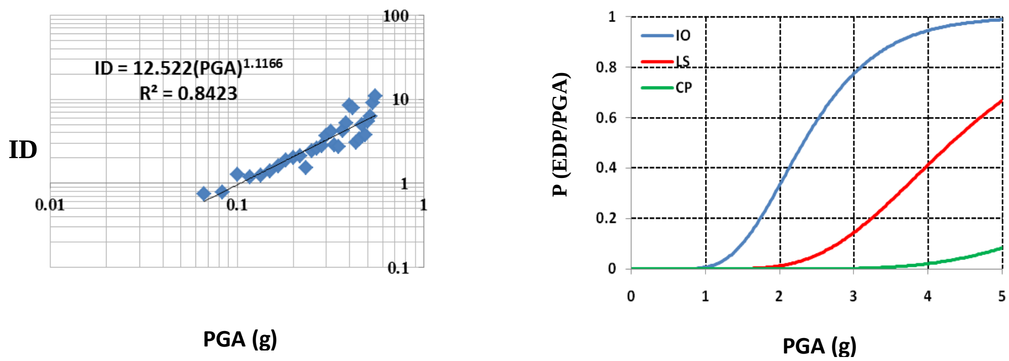
3.8.2 Fragility curves for Open Ground Storey building frames (considering EDP as inter-storey drift at ground storey)

The fragility curves are developedconsidering EDP as inter-storey drift at ground storey from the PSDM model as per the methodology explained in the previous sections, for three performance levels such as IO, LS and CP. The PSDM models and the corresponding fragility curves obtained for 10 storey 6 bay frame is presented in Figures 4.5 to 4.10.It is observed that the bare frame is the most fragile out of all the frames considered. The PGA increases the conditional probability of exceedance of the inter-storey drift increases.



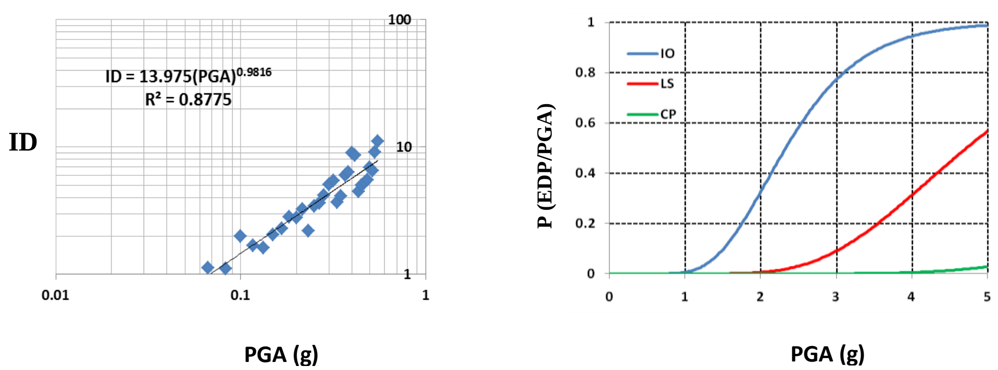
(a) **(b)**

Figure 3.7(a) Probabilistic Seismic Demand Models (b) Fragility Curves of 10 Storey 6 Bay Bare Frame



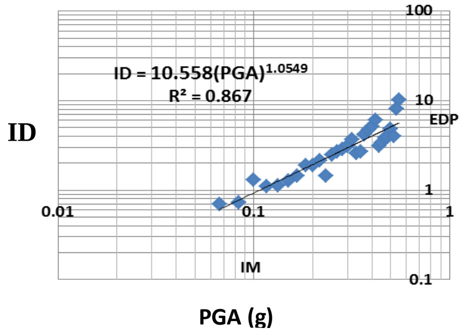
(a) **(b)**

Figure 3.8 (a) Probabilistic Seismic Demand Models (b) Fragility Curves of10 Storey 6 Bay Fully Infill Frame

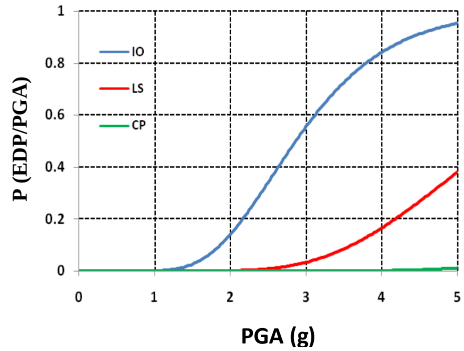


(a) **(b)**

Figure 3.9 (a) Probabilistic Seismic Demand Models (b) Fragility Curves of 10 Storey 6 Bay OGS 1.0 Frame

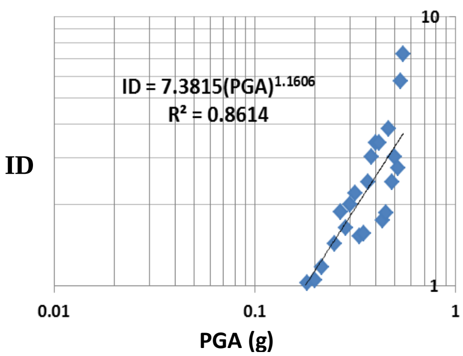


(a)

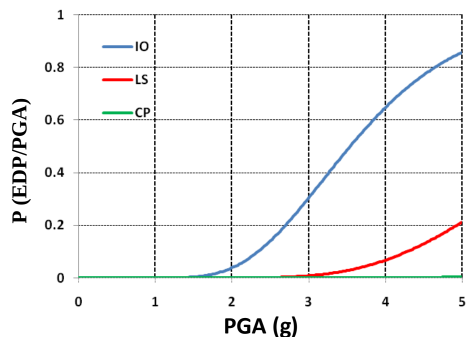


(b)

Figure 3.10 (a) Probabilistic Seismic Demand Models (b) Fragility Curves of 10 Storey 6 Bay OGS 1.0 Frame

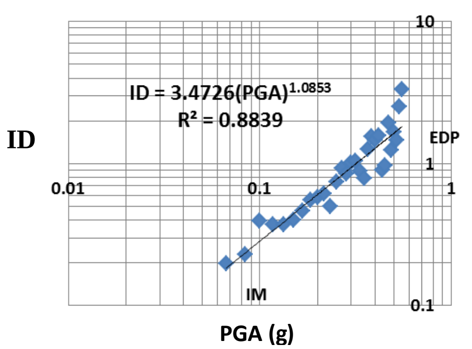


(a)

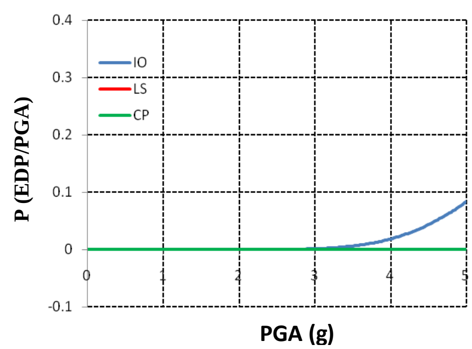


(b)

Figure 3.11 (a) Probabilistic Seismic Demand Models (b) Fragility Curves of 10 Storey 6 Bay OGS 2.0 Frame



(a)



(b)

Figure 3.12 (a) Probabilistic Seismic Demand Model (b) Fragility Curves, of 10 Storey 6 Bay OGS 2.5Frame

3.8.3 Fragility curves for Open Ground Storey building frames (considering EDP as inter-storey drift at various storeys)

The application of MF in the ground storey may reduce the inter-storey drift at ground but may increase for adjacent storeys. In order to study this effect, fragility curves are developed for OGS buildings considering EDP as maximum inter-storey drift at different storeys. Figure 3.13 presents the fragility curves of the building frames for different storeys for a 10 storey 6 bay bare frame building. It is observed that the second storey and first storey is fragile compared to ground storey. The same pattern is followed in all the performance levels except that the difference between the fragilities is increasing in the order for IO, LS and CP.

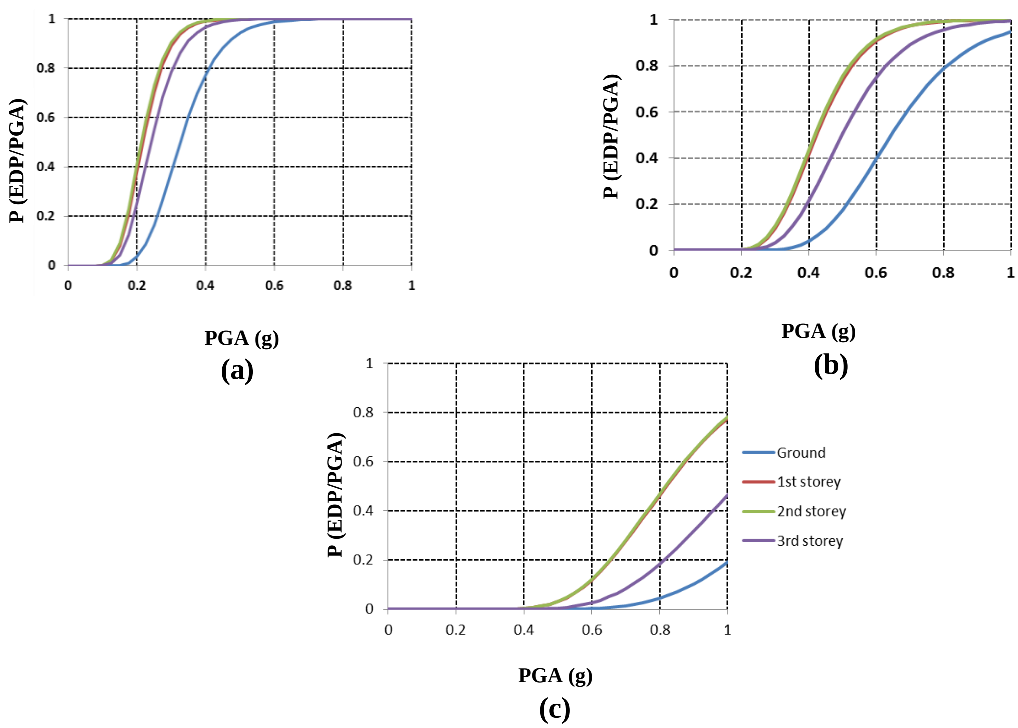


Figure 3.13 Fragility curves for different storeys for 10 Storey 6 bay Bare frame for performance levels (a) IO (b) LS (c) CP

Figure 3.14 presents the fragility curves of the 10 storey 6 bays FF frame building for different storeys. It can be seen that the ground storeyis more fragile compared to all the other storeys. The order of fragilities decreases in the order ground, first, second and third storeys. The same pattern is followed in all the performance levels, IO, LS and CP.

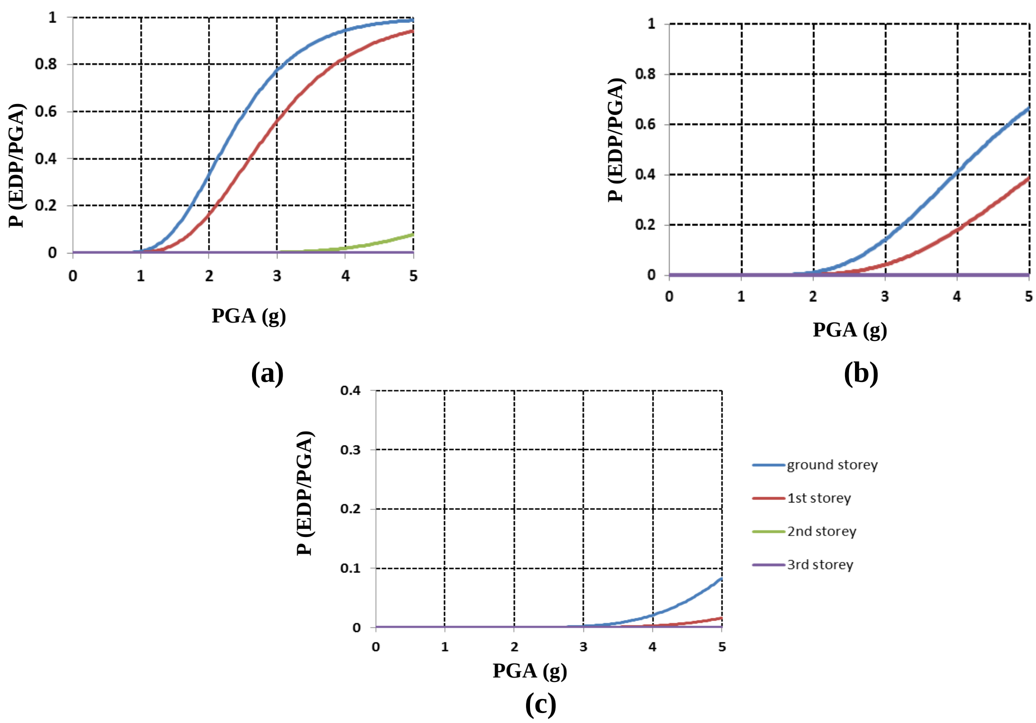


Figure 3.14 Fragility curves for different storeys for 10 Storey 6 bay FF frame for performance levels (a) IO (b) LS (c) CP

Figure 3.15 presents the fragility curves of the 10 storey 6 bayOGS1.0 frame building for different storeys. It can be seen that the ground storeyis more fragile compared to all the other storeys. The difference between fragility of ground storey compared to other storeys is much wider than observed in FF frame. This building represents the case of a large number of existing OGS buildings designed ignoring the MF. This case is an extremely vulnerable situation of an OGS frame that should be avoided. The same trend is followed in all the performance levels, IO, LS and CP.

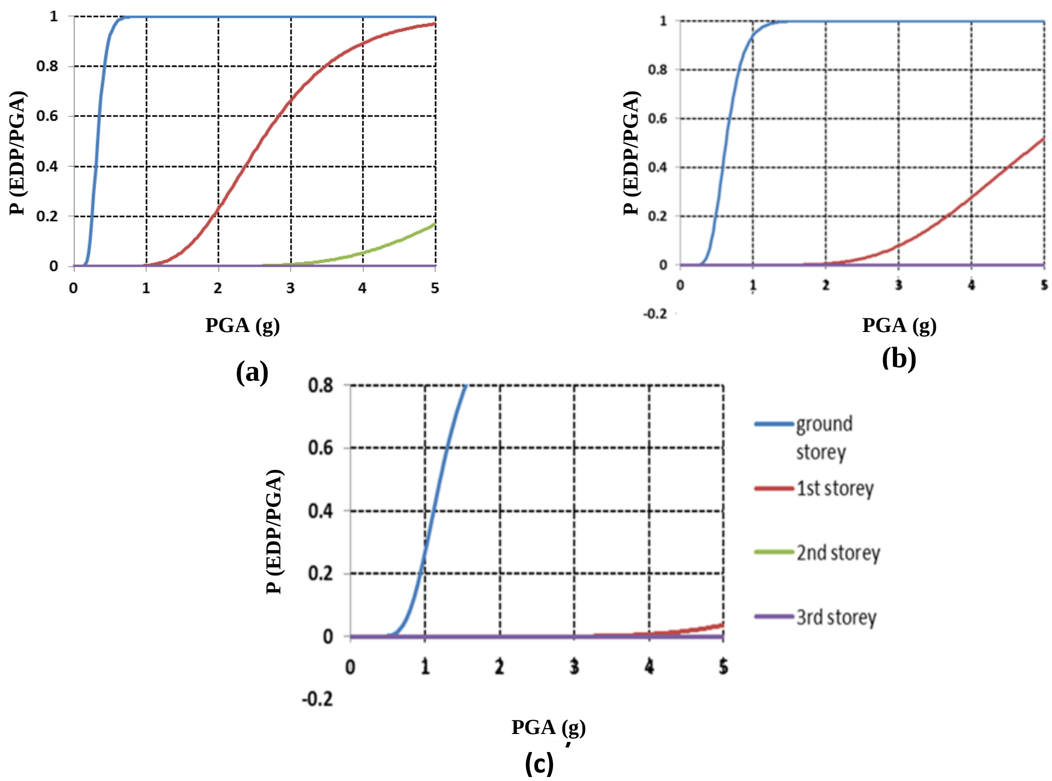


Figure 3.15 Fragility curves for different storeys for 10 Storey 6 bay OGS1.0 frame for performance levels (a) IO (b) LS (c) CP

Figure 3.16 presents the fragility curves of the 10 storey 6 bayOGS1.5 frame building for different storeys. It can be seen that the first storey is more fragile compared to all the other storeys. The ground storey became safer compared to first storey when MF increased from 1.0 to 1.5. The exceedance probability of inter-storey drift at ground storey is reduced by 25% at a PGA of 3g. This is perhaps due to the reduction of inter-storey drift at ground storey. The same trend is followed in all the performance levels, IO, LS and CP.

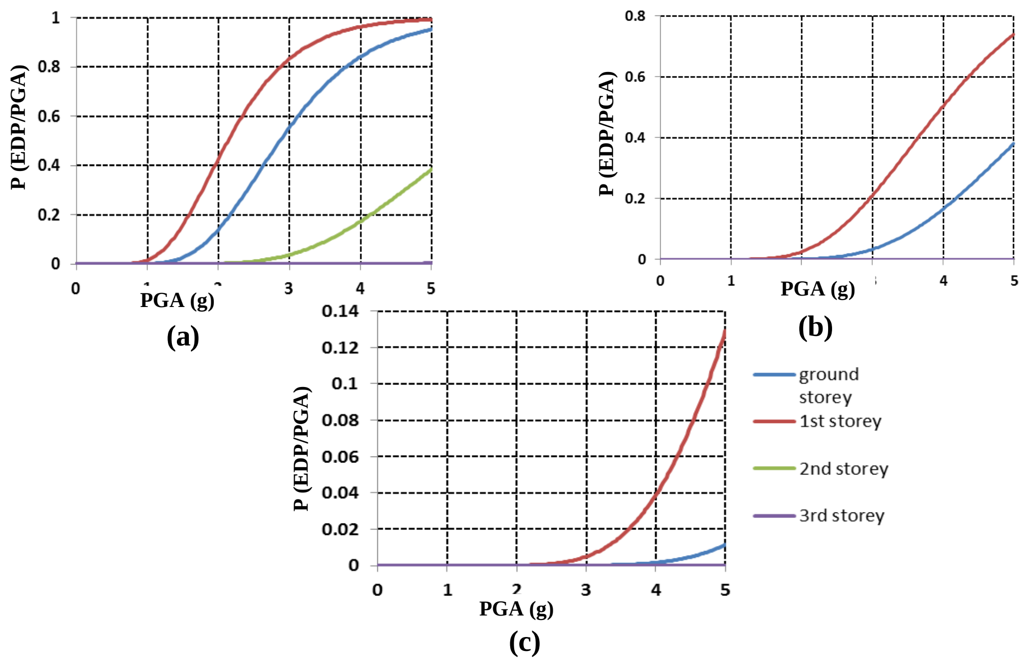


Figure 3.16 Fragility curves for different storeys for 10 Storey 6 bay OGS1.5 frame for performance levels (a) IO (b) LS (c) CP

Figure 3.17 shows the fragility curves of the 10 storey 6 bay OGS2.0 frame building for different storeys. It can be seen that the first storey is more fragile compared to all the other storeys as observed the case of MF =1.5. The ground storey became more compared to first storey when MF increased from 1.0 to 1.5. The exceedance probability of inter-storey drift at ground storey is reduced by 70% at a PGA of 3g. It may be due to the reduction of inter-storey drift at ground storey. The same trend is followed in all the performance levels, IO, LS and CP.

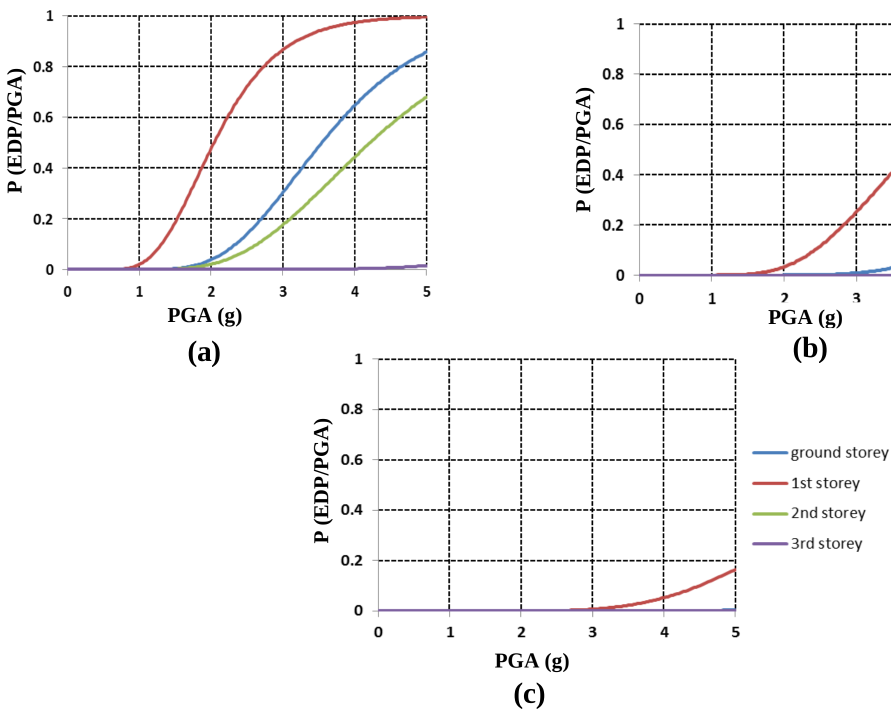


Figure3.17 Fragility curves for different storeys for 10 Storey 6 bay OGS2.0 frame for performance levels (a) IO (b) LS (c) CP

Figure 3.17 shows the fragility curves of the 10 storey 6 bay OGS2.5 frame building for different storeys. It can be seen that as the MF increased from 2.0 to 2.5, the ground storey is found to be safer than both first and second storey. The exceedance probability of inter-storey drift at ground storey is reduced by about 100% at a PGA of 3g. It may be due to the reduction of inter-storey drift at ground storey. The same trend is followed in all the performance levels, IO, LS and CP.

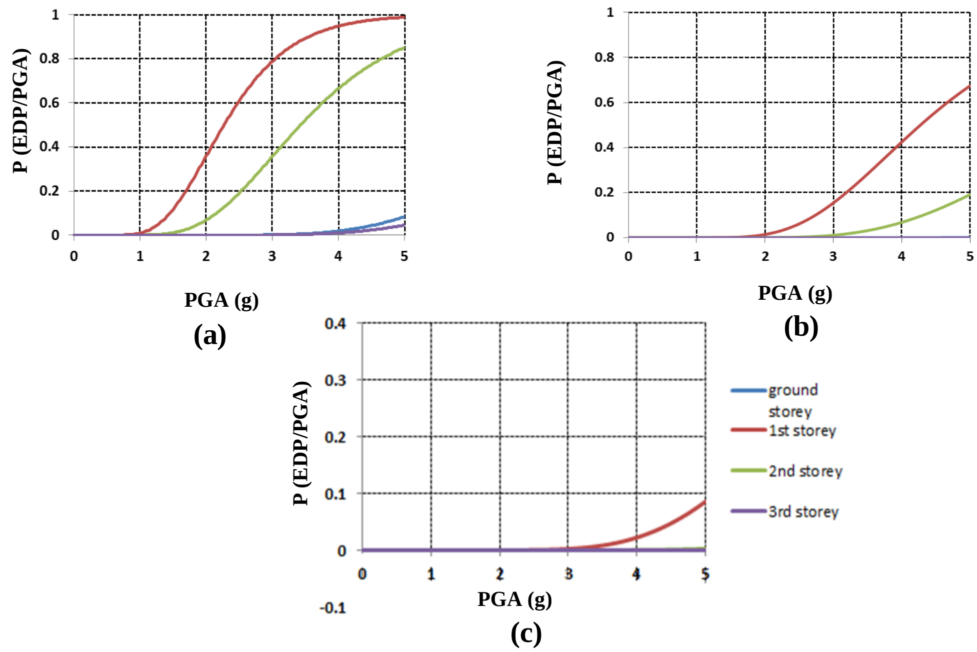


Figure 3.18 Fragility curves for different storeys for 10 Storey 6 bay OGS2.5 frame for performance levels (a) IO (b) LS (c) CP

Table 3.6 Most fragile storeys from Fragility Analysis

Frame	**Most fragile storey**	**Ground storey compared to most fragile storey**
Bare Frame	Second	55% less
Full Infilled frame	Ground Storey	0%
OGS 1.0	Ground Storey	0%
OGS 1.5	First Storey	25% less
OGS 2.0	First Storey	70% less
OGS 2.5	First Storey	100% less

Fragility curves for BF, FF, OGS-1, OGS1.5, OGS2 and OGS-2.5 buildings for three performance levels namely, IO, LS and CP are generated. The variation of exceedance probability of the inter-storey drift with the PGA is shown in Figure 3.19. The bare frame (BF) is found to be more vulnerable than the FF and OGS frame for all three performance levels considered. The OGS buildings designed by magnification factors 1.5, 2 and 2.5 are safer than that of FF in all the cases. The magnification factor 2.5 is likely to increase the performance than actually needed by decreasing the inter-storey drift. The same behaviour is observed in the case of eight and six storied frames.

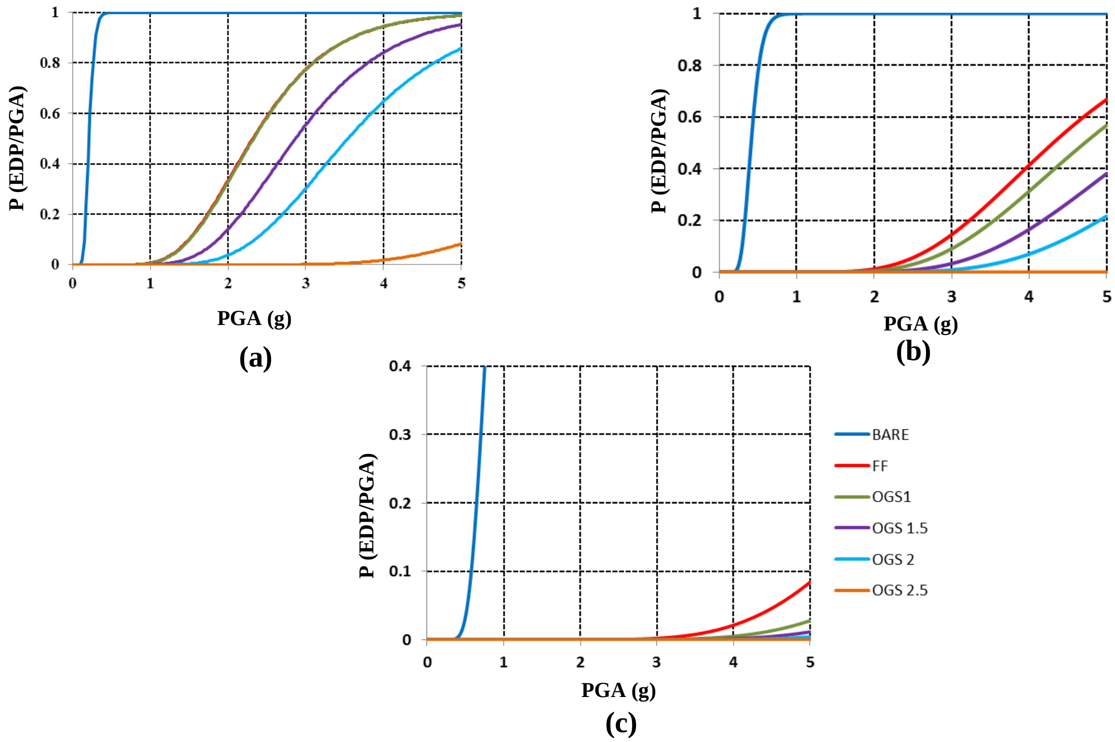


Figure 3.19 Fragility curves for 10 Storey 6 bay building frame for various cases at (a) IO (b) LS (c) CP, levels

3.9 PERFORMANCE OF FRAMES WITH STEPPED IRREGULARITIES

3.9.1 PSDM models for Building frames with stepped irregularities

The parameters, ‘a’ and ‘b’ of the PSDM models obtained for all the frames are summarized in the Table 3.7. A comparison of PSDM models for 10 storeyed building case study for all the infill wall configurations are drawn in a log-log graph as shown in the Figure 3.20. It can be seen that the inter storey drifts for frames without infill walls (BF, ST1 and ST2) are significantly higher than frames with infill walls (FF, STFF1 and STFF2). The inter-storey drifts of vertically irregular buildings designed with various stepped configurations without infill walls (ST1, ST2) are only marginally different. The same behavior is observed in the case vertically irregular buildings with infill walls (STFF1, STFF2).

Table 3.7 Parameters of Probabilistic Seismic Demand Models for 10, 8 and 6 storeyed for various infill walls configurations for stepped type buildings

Building types	**10 Storey 6 Bay**		**8 Storey 6 Bay**		**6 Storey 6 Bay**	
	a	**b**	**a**	**b**	**a**	**b**
Bare frame with single step without infill (ST1)	105.16	1.06	68.07	0.86	125.90	1.18
Bare frame with double step without infill (ST2)	74.57	0.92	84.41	1.16	93.20	1.13
Bare frame with single step with fully infill (STFF1)	8.95	0.88	10.89	1.02	14.11	1.16
Bare frame with double step with fully infill (STFF2)	9.75	0.93	12.23	1.11	11.11	1.79

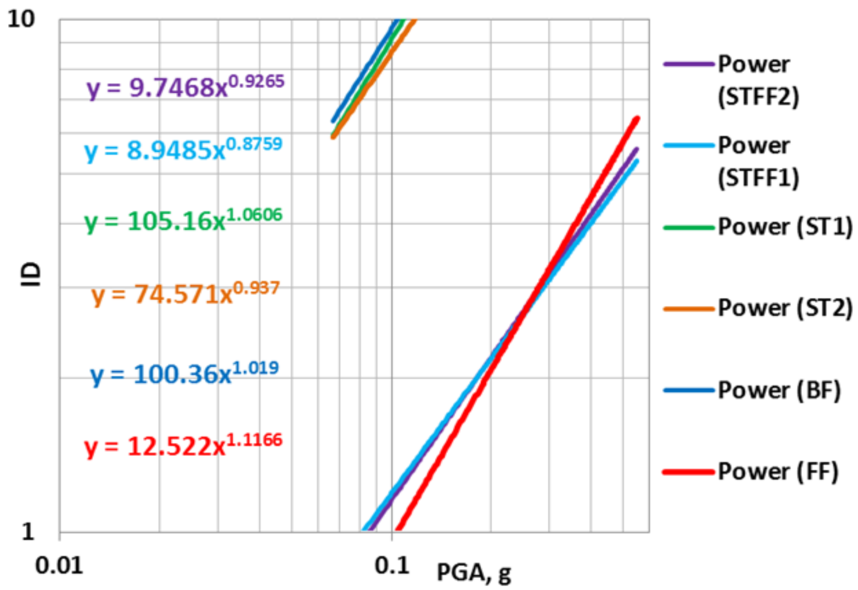
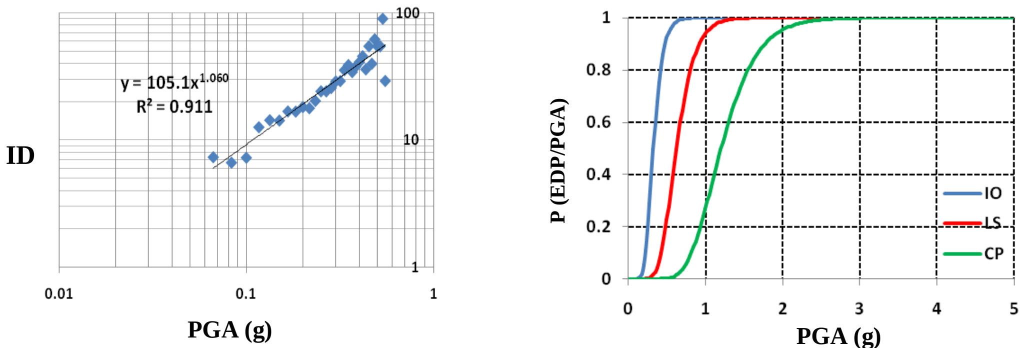
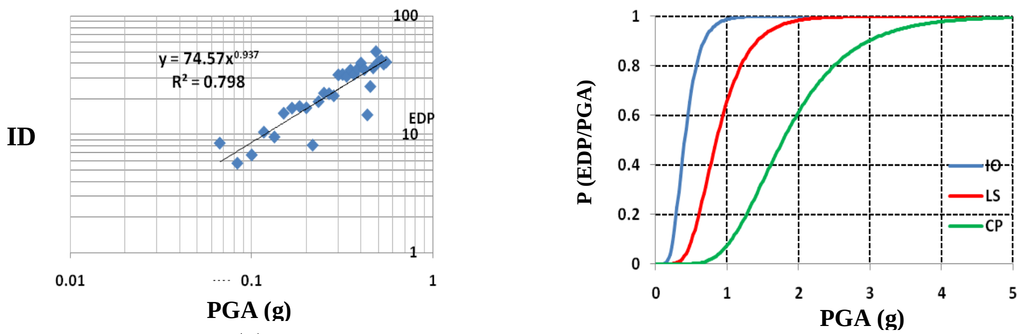


Figure 3.20 Comparison of PSDM models for various frames with stepped geometry



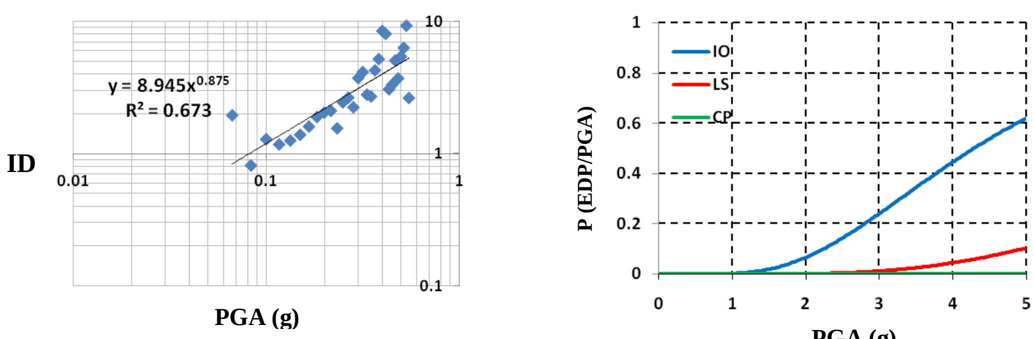
(a) **(b)**

Figure 3.21 (a) Probabilistic Seismic Demand Model (b) Fragility Curves, of 10 Storey 6 Bay single stepped Frame without infill wall (ST1)



(a) **(b)**

Figure 3.22 (a) Probabilistic Seismic Demand Model (b) Fragility Curves, of 10 Storey 6 Bay double stepped Frame without infill wall (ST2)



(a) **(b)**

Figure 3.23 (a) Probabilistic Seismic Demand Model (b) Fragility Curves, of 10 Storey 6 Bay single stepped Frame with infill wall (STFF1)

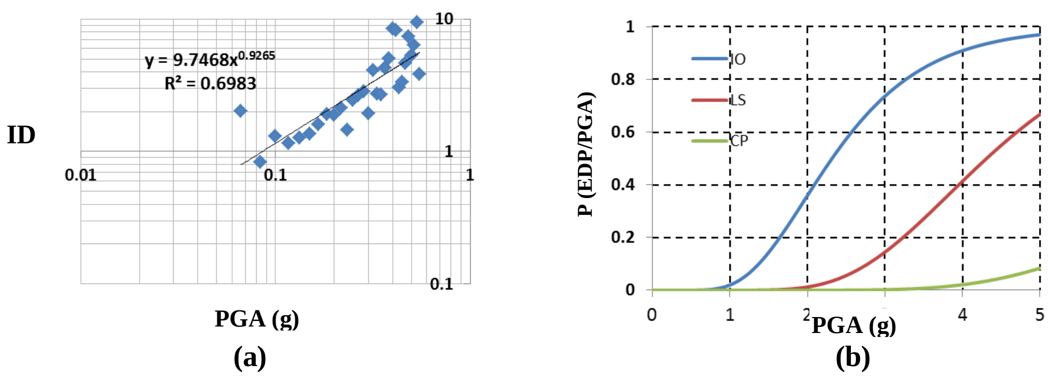


Figure 3.24 (a) Probabilistic Seismic Demand Model (b) Fragility Curves, of 10 Storey 6 Bay double stepped Frame with infill wall (STFF2)

Fragility curves for BF, FF, ST1, ST2, STFF1 and STFF2 buildings for three performance levels namely, IO, LS and CP are plotted. The variation of exceedance probability of the inter-storey drift with the PGA is shown in Figure 3.25. The frames without infill walls (BF, ST1 and ST2) are significantly fragile than that of frames with infill walls. The vertically irregular buildings with single and double stepped type without infill walls are safer than a bare frame. The vertically irregular building with single and double stepped type with infill wall is safer than that of FF and all other type of building considered for all the cases. As some of the frames are not present in the stepped buildings at top, compared to a FF frame, the mass and hence the inertia forces acting at top storeys would be less. This may be the reason for the marginally good behaviour observed in the case of vertically irregular buildings.

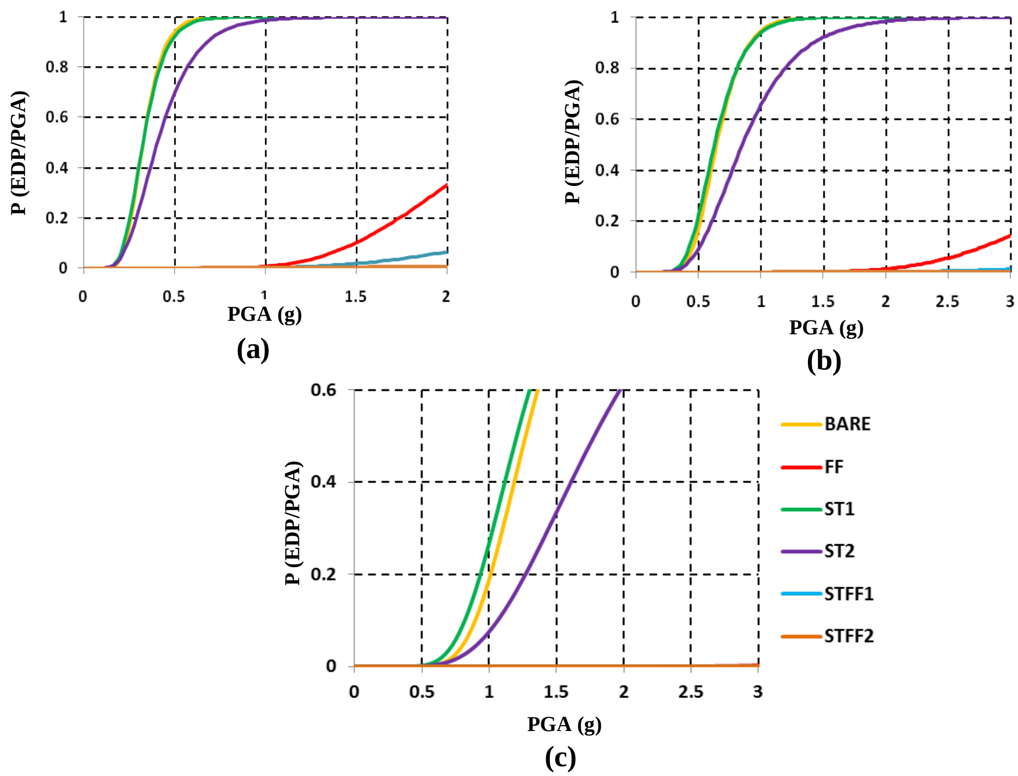


Figure 3.25 Fragility curves for different stepped configuration for 10 Storey 6 bay at performance levels (a) IO (b) LS (c) CP

3.10 SUMMARY

The performance of typical OGS buildings and vertically irregular buildings with stepped geometry is studied using fragility curves developed as per Cornel (2002). Uncertainties in concrete, steel and masonry are considered. The typical OGS buildings are designed considering various magnification factors and infill wall configurations.

Probabilistic seismic demand models (PSDM) are developed for all the frames considered for the analysis. The maximum inter-storey drift decreases as the MF increases. Inter-storey drift of bare frame is found to significantly higher that FF and OGS frames. The maximum inter-storey drift of ground storey of OGS frame decreases by 16% compared to FF when it is designed for a MF of 1.5. For a MF of 2.5, the inter-storey of ground storey is reduced by 50% compared to that of OGS buildings designed an MF of 2.0.Similarly, the maximum inter-storey drift reduction in an OGS building designed with MF of 2.0 compared to that of MF of 1.5 is found to be about 33%.

The fragility curves are developed for ground, first, second and third storey to find out the most vulnerable storey for each building considered. It is observed that for a bare frame second storey is more fragile and it is 55% more fragile than ground storey. In the case of fully infilled frame and OGS1.0, the ground storey is found to be more vulnerable that other storeys. As the MF increases from 1.0 to 1.5 or more than 1.5 (2.0 and 2.5), the ground storey becomes safer. In all three cases for MF = 1.5, 2.0 and 2.5, the first storey is more fragile compared to ground storey by 25%, 70% and 100% respectively.

Out of all the types of frames considered, the bare frame (BF) is found to be more vulnerable for all performance levels. The OGS buildings designed by magnification factors 1.5, 2 and 2.5 are safer than that of FF in all the cases.

The PSDM models of vertically irregular buildings show that the inter storey drifts for frames without infill walls (BF, ST1 and ST2) are significantly higher than frames with infill walls (FF, STFF1 and STFF2). From the fragility curves of the vertically irregular buildings it is observed that the stepped frames are found to be marginally safer than corresponding regular frames.

The fragility curves developed in the present Chapter is used to find the reliability index of the building frames, and is explained in the next Chapter.

CHAPTER 4

RELIABILITY ASSESSMENT OF RC FRAMES

4.1 INTRODUCTION

The fragility curves derived so far represent the probability that the maximum inter-storey drift in the frames will exceed inter-storey drift capacity corresponding to a particular performance level, if subjected to earthquake of given intensity in terms of effective PGA. In order to estimate the actual probability of failure and the reliability, which is inversely related to probability of failure, the fragility curves shall be combined with seismic hazard curve at the region selected in the study. The hazard curve should adequately represent the seismicity of the particular area for which the structure has been designed. For the present study, hazard curves of the Manipur region is selected, comes under seismic zone v, for the building is also designed. Hazard curve of a site, where an earthquake of 1.05g would be associated with approximately 2500 year return period or 2% probability of exceedance in 50 years. The probability of failure of the structure is found out by numerical integration. The reliability index is calculated as the inverse of the standard normal distribution. ISO 2394 (1988) recommends the Target Reliability Indices requirement for each performance level (consequences of failure) for each relative cost of measures. Target reliability values as per ISO 2394: 1988 are chosen for the present study to assess the reliability.

4.2 ASSESSMENT OF SEISMIC RELIABILITY FOR DIFFERENT HAZARD SCENARIOS

The fragility curves developed in the previous Chapter shall be combined with the hazard curve of the region for which the building is designed. Seismic hazard P [A = a], is described by the annual probabilities of specific levels of earthquake motion. In this study, hazard curve developed for Manipur is selected. Limit state probabilities can be calculated by considering a series of (increasingly severe) limit states, LS_i, through the expression:

$$P[LS_i] = \sum_a P[LS_i \mid A = a] P[A = a] \tag{4.1}$$

According to Cornell et. al (2002) A point estimate of the limit state probability for state *i* can be obtained by convolving the fragility $F_R(x)$ with the derivative of the seismic hazard curve, $G_A(x)$, thus removing the conditioning on acceleration as per Eq. (4.1).

$$P[LS_i] = \int F_R(x) \frac{dG_A}{dx} dx \tag{4.2}$$

The probability of failure is evaluated by numerical integration of Eq. 4.2. The numerical integration is explained graphically in the Figure 4.1. The hazard curve and the fragility curve are divided into small strips parallel to vertical axis. The slope of the hazard curve is multiplied by the ordinate of the fragility curve for each strip, and the summation of all the strips is carried out to evaluate the probability of failure.

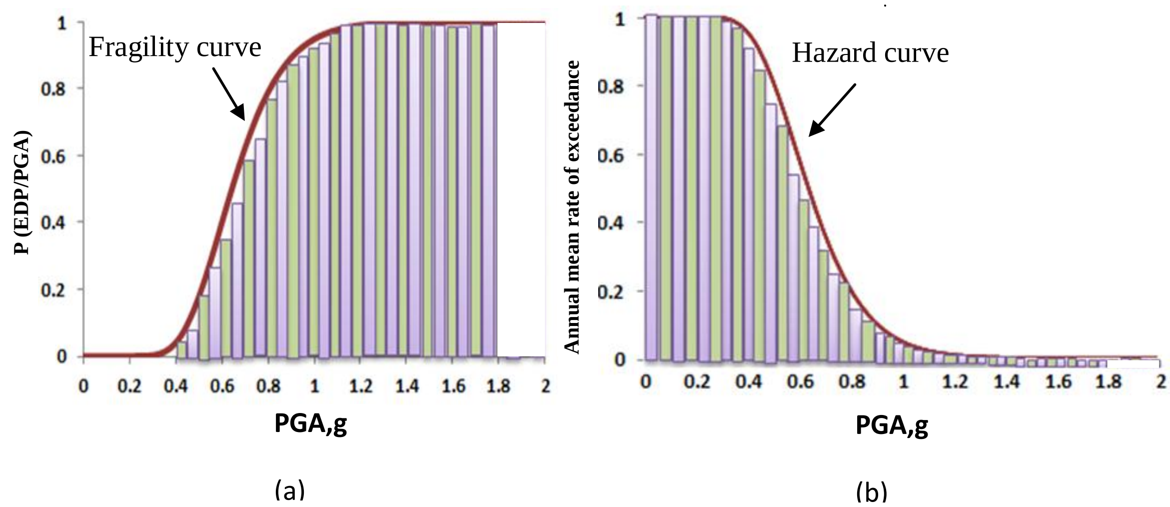


Figure 4.1 Numerical integration of a (a) fragility curve & (b) hazard curve for probability of failure

The parameters at the fragility-hazard interface must be dimensionally consistent for the probability estimate to be meaningful. The reliability index for corresponding probability of failure can be found by the following standard Equation.

$$\beta = -\phi^{-1}(pf) \tag{4.3}$$

Φ^{-1} is the inverse standard normal distribution.

4.3 SEISMIC HAZARD ANALYSIS

The seismic hazard at a building site is displayed through a complimentary cumulative distribution function (CCDF), as per. The hazard function is the annual frequency of motion intensity at or above a given level, *x*, to the intensity. Elementary seismic hazard analysis shows that at moderate to large values of ground acceleration, there is a logarithmic linear relation between annual maximum earthquake ground acceleration or spectral acceleration, and the probability, $G_A(a)$, that specifies values of acceleration are exceeded, reference. This relationship implies that A is described by following equation,

$$G_A(x) = 1 - \exp[-(x/u)^{-k}] \tag{4.4}$$

u and *k* are parameters of the distribution. Parameter *k* defines the slope of the hazard curve which, in turn, is related to the coefficient of variation (COV) in annual maximum peak acceleration.

A methodology for the assessment of seismic risk of building structures is presented by Pallav et.al. (2012). The hazard analysis is an estimated probability of exceedance of a corresponding to certain of ground motion in 50 years. The hazard depends on the magnitudes and locations of likely earthquakes, how often they occur, rocks properties and sediments that earthquake waves travel through etc.

Since Manipur, located at North east part of India, which is a seismically active region, the probabilistic seismic hazard curve of Manipur is selected for the present study. This curve is developed by Pallav et al (2012). The hazard curve of the Manipur region is shown in the Figure 4.2.

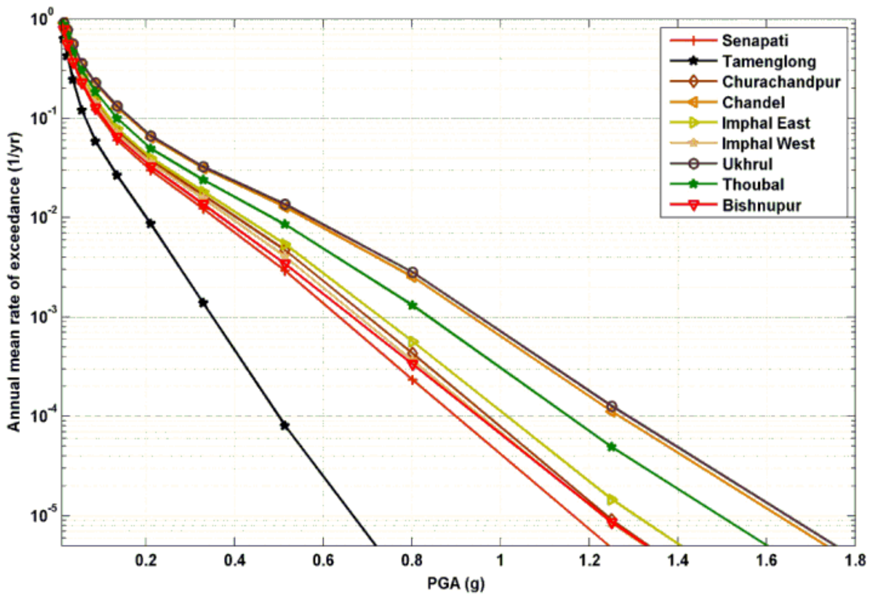


Figure 4.2 Hazard curves for different region of Manipur region (Pallav et. al., 2012)

From the graph shown in Figure 4.2, the hazard curve of Ukhrul location is extracted and plotted as shown in Figure 4.3.

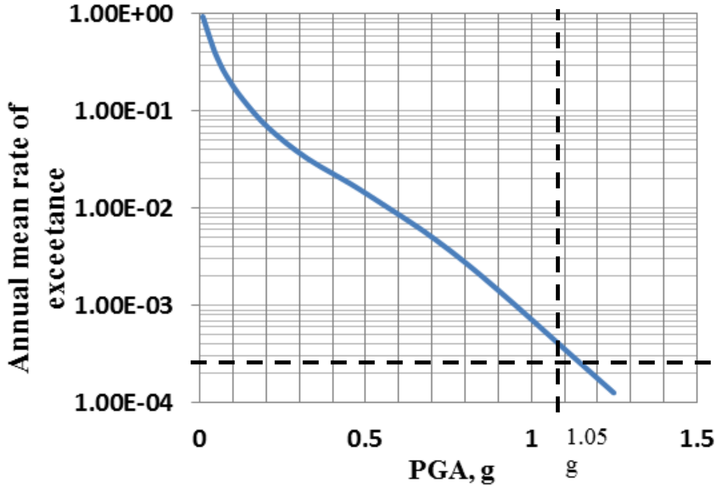


Figure 4.3 Numerical integration of a hazard curve for probability of failure

4.4 ASSESSMENT USING THE RELIABILITY I NDICES

The reliability index is estimated from the fragility curves as per the procedure explained previous section. The reliability index is calculated for each PGA, which will yield reliability indices corresponding to each PGA. In order to check the target reliability to be achieved by the building frames for various PGAs and performance levels, target reliabilities using some acceptable standards are to be selected. In the present study, Target Reliability Indices in accordance with ISO 2394 (1998) is used and is shown in Table 4.1. This table shows the target reliability requirement for each performance level (consequences of failure). The assessment of performance of each building is carried out by comparing the reliability indices obtained for each building with corresponding target reliability indices corresponding to moderate level of consequences of failure. In order to assess the performance of the buildings at collapse prevention, the target reliability indices is taken as 3.8.

Table 4.1 Target reliability Index in accordance with IS 2394 (1998)

Relative Cost of Measures	**Consequences of Failure**		
	Some **IO**	Moderate **LS**	Great **CP**
High	**1.5**	**2.3**	**3.1**
Moderate	**2.5**	**3.1**	**3.8**
Low	**3.1**	**3.8**	**4.3**

Variation of reliability index (β) with the parameter, PGA is plotted for 8storey 6bay frames in the Figure 4.4 and 4.5 for OGS frames and stepped irregular frames respectively. It is observed that as the PGA increases the reliability index decreases. Target reliability suggested by ISO 2394 (1998) for moderate building with severe damage is marked as 3.8 in the Figure 4.4. PGA corresponding to 2% probability of occurrence in 50 years is found to be 1.05g from the hazard curve of Manipur region (shown in Figure 4.4 and 4.5). Figure 4.4 show that Reliability indices obtained for the bare frame building (designed as per the Indian Standards) at the PGA of 1.05g is 3.27. It is found that the bare frame is failed to achieve the target reliability of 3.8 at the PGA of 1.05g which corresponds to 2% probability of occurrence in 50 years, in Manipur location. The OGS frames (modeled with stiffness and strength of infill walls) achieved a reliability of more than target reliability (3.8), at PGA of 1.05g.

Figure 4.5 shows the variation of reliability indices for various PGAs for 8storey 6 bay vertically frames with stepped configurations and infill wall arrangements. All the bare frames are failed to achieve the target reliability requirement at a PGA of 1.05g. Presence of infill walls is more important even in stepped vertically irregular buildings to achieve the target reliability.

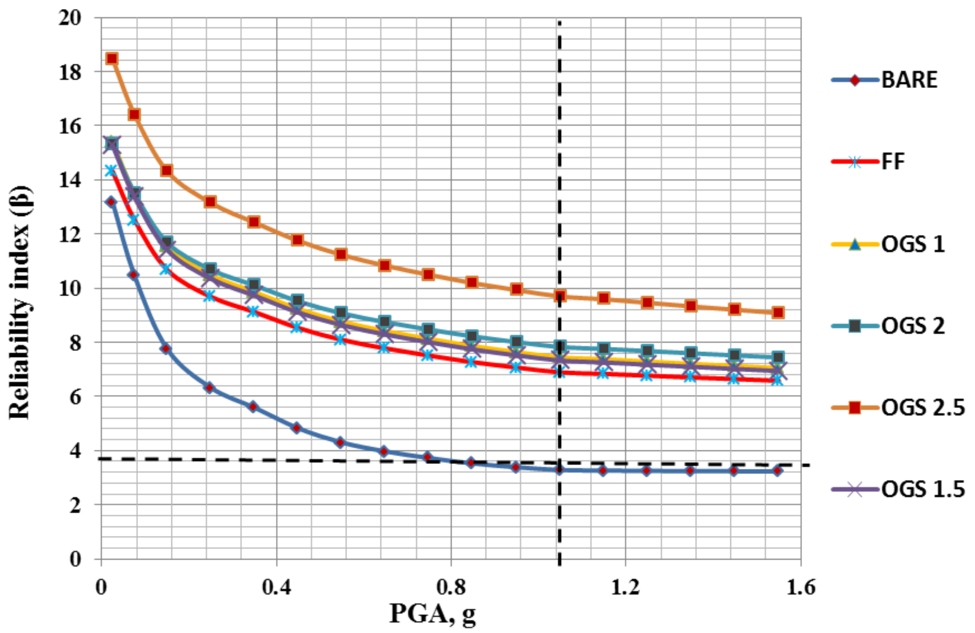


Figure 4.4 Reliability Curves for 8 storey 6 bay OGS frames for CP performance levels

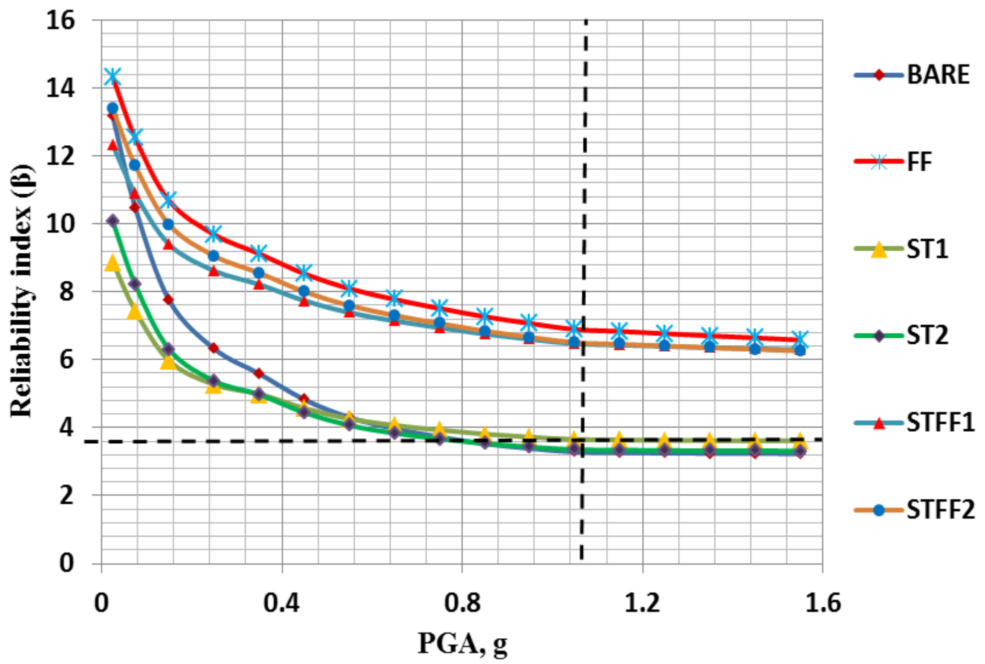


Figure 4.5 Reliability Curves for 8 storey 6 bay stepped irregular frames for CP performance levels

The probability of failure and reliability indices for all the frames at PGA of 1.05g is calculated for 6, 8 and 10 storeyed OGS frames at different performance levels. These are presented in the Table 4 1.

Among all frames, bare frames are found to be more vulnerable due to higher values of failure probability. The stiffness and strength of infill walls are neglected in the bare frame analysis and the force demands in the bare frame is high and hence they are more vulnerable. In reality the infill walls will contribute stiffness and strength to the building, which increases the performance of the building.

From Table 4.2, it can be seen that Bare frames (BF) are not able to meet the target reliability suggested by ISO 2394 1998 in all the performances levels where as the full infilled frames (FF) meets the target reliability in all performances levels.

The infill walls are ignored at analysis and design stage, in the current design methodology. In reality, the infill walls which is ignored and provided at the time of construction, contribute to some stiffness and strength to the global performance of the buildings (e.g. fully infilled frames).

However, for an Open ground storey building the same design methodology may not guaranty the required performance. However in the present study OGS1 marginally reaches the Target Reliability in all the performance levels, which may not be always true. This implies that more research is required in this direction. For OGS 2.5 Reliability Indices are found to be twice that of target reliability, which indicates that the factor MF may be more conservative. For optimum design of an OGS building, particularly for the design magnification factor, the target reliability can be a considered as a basis.

Table 4.2 PROBABILITY OF FAILURE AND SEISMIC RELIABILITY OF THE FRAMES FOR EACH LIMIT STATES FOR MANIPUR REGION

Type		**6S6B**		**8S6B**		**10S6B**	
		Pf	**β**	**Pf**	**β**	**Pf**	**β**
Bare	CP	0.0033	2.71	6.06E-04	3.23	5.31E-04	3.27
FF	CP	2.35E-11	6.58	2.34E-11	6.58	2.34E-11	6.58
OGS 1	CP	7.46E-11	6.40	9.09E-13	7.04	7.30E-13	7.07
OGS 1.5	CP	7.18E-12	6.75	1.96E-12	6.94	6.04E-14	7.41
OGS 2	CP	5.47E-12	6.79	5.00E-14	7.44	2.85E-15	7.81
OGS 2.5	CP	3.18E-11	6.53	5.32E-20	9.08	5.13E-25	10.2

4.5 SUMMARY

From the fragility curves developed in the previous chapters, Reliability indices are calculated by combining the fragility curves with the seismic hazard of the Manipur Region, where the building frames are assumed to be located. From the hazard curve, the PGA corresponding to 2% probability exceedance of 50 years is selected to evaluate the reliability index. The reliability indices calculated for each frames, (OGS frames and stepped irregular frames) are compared against the target reliability suggested by ISO standard. It is found that the bare frames are failed to achieve the target reliabilities. This implies that the inclusion of infill walls in the analysis improves the performance of the frames significantly under seismic loads.

CHAPTER-5

CONCLUSIONS AND FUTURE SCOPE

5.1 SUMMARY

The buildings with vertical irregularity are very common in Indian construction due to its functional advantages. Open ground storey (OGS) is an example of an extreme case of vertically irregularity. These types of buildings are found to be the most affected in an earthquake as seen from the past Indian earthquakes. The performance of typical OGS buildings and vertically irregular buildings with stepped geometry are studied, with considering the Uncertainties in material properties. The behaviour of typical OGS buildings designed by considering various magnification factors and the stepped type irregularity with various infill wall configurations are observed for different performance levels is observed.

5.2 CONCLUSIONS

The conclusion of the study is categorised into two parts. In the first part the behaviour of OGS Buildings are explained. And the stepped type buildings performances are mentioned in the second part.

5.2.1 OGS buildings

- The probability of exceedance and fragility curves and drawn for all the frames at is calculated for 6, 8 and 10 storeyed OGS frames with different MFs at different performance levels as IO. LS and .CP.

- Probabilistic seismic demand models (PSDM) are developed for all the OGS frames considered for the analysis using log-log graph. A comparison of PSDM models for all the building case studies with various infill wall configurations are plotted. The fragility curves are developed considering EDP as inter-storey drift at ground storey. From the PSDM model as per the methodology explained in the previous sections, for three performance levels such as IO, LS and CP.

- From the fragility curves it is observed that the bare frame is the most fragile out of all the frames considered. The PGA increases the conditional probability of exceedance of the inter-storey drift increases. For OGS buildings the maximum inter-storey drift are found to be decrease as the increase of MF. Among the all buildings the Inter-storey drift of bare frame (BF) is found to significantly higher that FF and OGS frames. The maximum inter-storey drift of ground storey of OGS frame decreases by 16% compared to FF when it is designed for a MF of 1.5. For a MF of 2.5, the inter-storey of ground storey is reduced by 50% compared to that of OGS buildings designed with MF of 2.0. Similarly, the maximum inter-storey drift reduction in an OGS building designed with MF of 2.0 compared to that of MF of 1.5 is found to be about 33%.

- Also the fragility curves are developed for ground, first, second and third storey to observe the most vulnerable storey for all the considered building. It is found that in case of bare frame, the second storey is the fragile and it is 55% more fragile than ground storey. In the case of fully infilled frame and OGS1.0, the ground storey is found to be more vulnerable that other storeys. As the MF increases from 1.0 to 1.5 or more than 1.5 (2.0 and 2.5), the ground storey becomes safer. In all three cases for MF = 1.5, 2.0 and 2.5, the first storey is more fragile compared to ground storey by 25%, 70% and 100% respectively.

- Out of all the types of frames considered, the bare frame (BF) is found to be more vulnerable for all performance levels. The OGS buildings designed by magnification factors 1.5, 2 and 2.5 are safer than that of FF in all the cases.

5.2.2 Stepped building

- The same procedure was adopted for the generating the fragility curves for stepped type building. From the PSDM models the vertically irregular building it can be concluded that the inter storey drifts for frames without infill walls (BF, ST1 and ST2) are significantly higher than frames with infill walls (FF, STFF1 and STFF2). The inter-storey drifts of vertically irregular buildings designed with various stepped configurations without infill walls (ST1, ST2) are only marginally different. From the fragility curves of the vertically irregular buildings it is observed that the stepped frames are found to be marginally safer than corresponding regular frames. The same behavior is observed in the case vertically irregular buildings with infill walls (STFF1, STFF2).

- The frames without infill walls (BF, ST1 and ST2) are significantly fragile than that of frames with infill walls. The vertically irregular buildings with single and double stepped type without infill walls are safer than a bare frame. The vertically irregular building with single and double stepped type with infill wall is safer than that of FF and all other type of building considered for all the cases. As some of the frames are not present in the stepped buildings at top, compared to a FF frame, the mass and hence the inertia forces acting at top storeys would be less. This may be the reason for the marginally good behaviour observed in the case of vertically irregular buildings.
- The vertically irregular building with single and double stepped type with infill wall is safer than that of FF and all other type of building considered for all the

cases. As some of the frames are not present in the stepped buildings at top, compared to a FF frame, the mass and hence the inertia forces acting at top storeys would be less. This may be the reason for the marginally good behaviour observed in the case of vertically irregular buildings.

5.2.3 Reliability analysis

- The present study is also focused on the seismic reliability assessment of typical vertically irregular building with various configurations. For the analysis Manipur region is chosen the hazard curve developed by pallav et.al. (2012) is considered which has plotted by considering different regions. From the entire region Ukhraul is selected for the analysis which is the worst case among all. The hazard curve is combined with the fragility curve to find the joint probability of failure and corresponding reliability.

From the reliability graph, the observations are explained below.

- The probability of failure and reliability indices for all the frames at PGA of 1.05g is calculated for 6, 8 and 10 storeyed OGS frames at different performance levels as IO , LS and .CP..

- Bare frames (BF) are not able to meet the target reliability suggested by ISO 2394 1998 in all the performances levels where as the full infilled frames (FF) meets the target reliability in all performances levels.

- However, for an Open ground storey building the same design methodology may not guaranty the required performance. However in the present study OGS1 marginally reaches the Target Reliability in all the performance levels, which may not be always true. This implies that more research is required in this direction. For OGS 2.5 Reliability Indices are found to be twice that of target reliability, which indicates that the factor MF may be more conservative. For optimum design of an OGS building, particularly for the design magnification factor, the target reliability can be a considered as a basis.

- The infill walls are ignored at analysis and design stage, in the current design methodology. In reality, the infill walls which is ignored and provided at the time of construction, contribute to some stiffness and strength to the global performance of the buildings (e.g. fully infill frames). So, further research work is required in this direction.

5.3 SCOPE OF FUTURE WORKS

- The present study is limited to reinforced concrete multi-storey framed buildings that are regular in plan and irregular in elevation. It can be extended to buildings having irregularity in plan. This involves analysis of three dimensional building frames that accounts for torsional effects. Also, similar studies can be carried out on steel framed buildings.

- In the analysis OGS buildings MFs are used upto 2.5 as per IS code. It can extend beyond 2.5 that can extend for different codes.

- Vertically irregular buildings with basement, shear walls and plinth beams are not considered in this study. The present methodology can be extended to such buildings also.

- Soil - structure interaction effects are neglected in the present study. It will be interesting to study the response of the Vertically irregular buildings considering the soil - structure interaction.

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